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16. ABSTRACT

As part of joint interests in environmental matters between the United States and Japan, a Conference on Sewage Treatment Technology is held at intervals of about 18 months. This publication contains papers from the Japanese group and from the American side that were presented at the Sixth Conference. Subject matter covered included nitrogen removal, sludge processing, instrumentation, sewage plant construction and water quality.

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FOREWORD

Environmental improvement is a worldwide need. Maintaining clean water supplies and managing municipal and industrial wastes is a vital element of a quality environment.

The participants in the United States-Japan cooperative project on sewage treatment technology have completed their sixth conference. These conferences, held at 18-month intervals, give the scientists and engineers of the cooperating agencies an opportunity to study and compare the latest practices and developments in the United States and Japan. These Proceedings of the Sixth Conference comprise a useful body of knowledge on sewage treatment, which will be available not only to Japan and the United States but also to all nations of the world who desire it.

Douglas M. Costle
Administrator

Washington, D.C.

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OFFICIAL OPENING OF SIXTH CONFERENCE, USEPA, CINCINNATI, OHIO

MR. JOHN J. CONVERY, NEWLY APPOINTED GENERAL CHAIRMAN OF THE JOINT U.S./JAPAN CONFERENCES ADDRESSING DELEGATES OF SIXTH CONFERENCE, WASHINGTON, D.C.

ALICE BRANDEIS POPKIN, ASSOCIATE ADMINISTRATOR FOR INTERNATIONAL ACTIVITIES GREETS JAPANESE DELEGATES AT SIXTH CONFERENCE, WASHINGTON, D.C.

DR. TAKESHI KUBO AND MR. ROBERT A. CANHAM, EXECUTIVE DIRECTOR, WATER POLLUTION CONTROL FEDERATION DISCUSS AREAS OF MUTUAL INTEREST AT SIXTH CONFERENCE, WASHINGTON, D.C.

JOINT COMMUNIQUE

SIXTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

WASHINGTON, D.C.
NOVEMBER 3, 1978

1. The Sixth United States/Japan Conference on Sewage Treatment Technology was held in Cincinnati, Ohio, and Washington, D.C., from October 30 through November 3, 1978.

2. The Japanese delegation headed by Dr. T. Kubo, Co-Chairman of the Conference, Vice President, Japan Sewage Works Agency, was composed of three National government officials and three local government officials.

3. Mr. Francis M. Middleton, formerly Senior Science Advisor, Municipal Environmental Research Laboratory, Cincinnati, Ohio, US EPA, served as General Chairman and Head of the Cincinnati delegation which consisted of six US EPA officials and three representatives of industry, local government and regional water basin commission.

Mr. John T. Rhett, Deputy Assistant Administrator for Water Program Operations was Head of the Washington delegation which consisted of three US EPA, one state and three local government officials. In addition to government officials, conference delegates included Mr. Robert A. Canham, Executive Director, Water Pollution Control Federation and Dr. Daniel P. Sheer, Planning Engineer, Interstate Commission on the Potomac River Basin.

4. Prior to the Conference, the Japanese delegation visited San Jose and Pomona Water Reclamation Plant, Joint Water Pollution Control Plant, Los Angeles County Sanitation Districts, R. L. Jackson Sewage Treatment Plant, Clayton County Water Authority, Dalton County Utility Authority, Georgia, Mill Creek Sewage Treatment Plant, City of Cincinnati, Ohio and Ohio River Valley Water Sanitation Commission. In the Washington, D.C. area, the treatment facilities in Arlington and Alexandria, Virginia, and the Occoquan Water Reclamation Plant, Upper Occoquan Sewage Authority were visited.

5. Principal topics of the Sixth Conference in Cincinnati were biological nitrogen control, wastewater and sludge research program, air pollution discharges from sewage sludge incinerators, several applications of pure oxygen activated sludge systems for treating mixtures of high organic industrial and domestic wastes, tannery waste treatment and progress in instrumentation and automation. Principal topics in the Washington Conference were: Institutional Structures of Water Pollution Control in Japan; Financing of Sewage Works Including Sewer User Charge System; Mission and Activities of Japan Sewage Works Agency at Present and Future; the 1977 Mid-Course Corrections" to the Federal Clean Water Program; Construction Grants Program--A Regional Perspective; Water Quality Programs of the Virginia State Water Control Board; Water Supply and Wastewater Management Planning in the Washington, D.C. Metropolitan Area; The Occoquan Watershed Policy--A Comprehensive Program to Save a Water Supply; and Pretreatment Standards: EPA Strategy to Curb Hazardous Industrial Discharges.

6. Progress reports of joint research works were presented from both sides during the conference. Data and findings on the joint research projects were mutually supportive and provided increased insights with the nature of the problem and potential solutions. This was especially true of metal emissions from incinerators and instrumentation performance.

7. A decision was made to continue the productive joint research efforts on municipal sludge disposal, agricultural use of sludge and instrumentation and automation. The fourth area of joint research, biological nitrogen control, is to be broadened to include biological removal of specific contaminants including toxic substances.

8. Recent engineer exchanges included a two-week visit to Japan by Mr. Richard I. Field, Chief of the Storm and Combined Sewer Section, Municipal Environmental Research Laboratory, Cincinnati, and a six-month visit to the U.S.A. by Mr. K. Tanaka and Mr. S. Hiromoto of the Japan Sewage Works Agency. The parties agreed in principle to the continuation of the Engineer Exchange Program. Two nominations for Japanese participation in the Program during 1979 were identified.

9. At the end of the Cincinnati Conference, a ceremony was held awarding an honor to Mr. F. M. Middleton for his outstanding contribution to the exchange program between the two countries since 1971.

10. Mr. John J. Convery, Director, Wastewater Research Division, MERL, was named to succeed Mr. Middleton as General Conference Chairman for the United States.

11. It was proposed by the Japanese side that the Seventh Conference shall be held in Japan about April 1980.

12. A Proceedings of the Conference will be printed in English and in Japanese.

JAPANESE PAPERS

PAPERS FOR THE SIXTH US/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

Cincinnati, Ohio

October, 30-31, 1978

Ministry of Construction
Government of Japan

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CHAPTER 1

BIOLOGICAL NITROGEN CONTROL

I. BIOLOGICAL NITROGEN CONTROL IN ACTIVATED SLUDGE PLANT (LABORATORY STUDY)

The objectives of the study are: 1) to determine physical, chemical and biological operational conditions for effective nitrification in the conventional activated sludge process. 2) to define optimum operational variables to maintain effective nitrification and organic removal at existing sewage treatment plants with the least modification of the facilities. 3) to develop recovery techniques of organic carbon from anaerobic digester supernatant, for carbon source of denitrification process.

Effect of pH and sludge retention time (SRT) on nitrification has been studied by the laboratory scale plug flow type aerators under the temperature of 20°C, using synthetic sewage with the substrate of dextrin, meat extract, yeast extract, peptone, ammonium sulfate and other inorganic components.

Effects of seawater, Mg, anions and phosphate buffer on the activities of the nitrifiers in activated sludge have been studied separately by a shaking culture method and an aeration method. Synthetic seawater used here was the one developed by Lyman et al. (1949). Determination of the nitrifiers was followed by IBP (1968).

1. SUMMARY

Nitrification in activated sludge process: 1) Result of the field survey showed that numbers of nitrifiers were in the range of 10^3 to 10^4 N/m (Dec. 1975) and Nitrobacter was more than Nitrosomonas. 2) Analysis of "Annual Sewage Treatment Plants' Report" indicated: i) Highest degree of nitrification under the temperature of 18 ~ 23°C. ii) Minimum effluent dissolved oxygen level of 28% of saturation value for effective nitrification. 3) Generation time of the each organisms at the temperature of 20°C in mixed culture system was; ammonium oxidizers 7.4 hours, nitrite oxidizers 11.6 hours, activated sludge ciliata 2.5 days, Rotifera 3.5 days and Sarcodina 13.6 days. 4) More than 10^4 N/ml of nitrifiers would be necessary to obtain nitrate in the effluent. 5) Nitrification activity of the biomass was not likely to be interferred with a shock load of 2 hours when pH was kept within 4 to 11. 6) Above nitrification activity is affected by organic loadings. When pH of the system goes down from 8, which, is an optimum pH level of nitrification, to the lower, effect of organic loadings become severe. The long SRT is necessary to obtain the same extent of nitrification.

7) As for the biological fauna, activated sludge ciliata and Rotifera were dominant when pH was within the range of 7 to 8 and Sarcodina was easily to appear when it goes down to 6. This tendency was much clearer when STR of the system was kept longer. 8) Rotifera and Sarcodina could be used as index organisms to determine whether nitrification could be expected or not in the system. 9) Optimum pH for nitrification was 8. To achieve substantial nitrification, SRT of

the system should be kept more than 3, 6, 11 days when pH is 8, 7, 6 respectively under the temperature of 20°C. 10) Removal efficiency of soluble organics was more than 97% in all the experimental runs regardless to the pH and SRT. High solid removal efficiency was obtained when SRT was maintained between 5 and 10 days. However, when SRT was less than 3 days, solid removal was deteriorated due to the bulking of the sludge and when SRT was more than 10 days, solid washout was observed due to the deflocculation and floatation of the denitrified sludge.

Effect of trace inorganics: 1) Growth of fresh water nitrifiers were inhibited when cultured with more than 40% of sea water, while maximum growth rate was obtained when no sea water was added at all.

2) Under the case that both sea water and fresh water nitrifiers were co-inhabited, maximum growth rate were obtained when cultured just in sea water. However, lag phase was shortest when sea water was added 40 to 60%. This may indicate that changes in "environment" of the nitrifying bacterias is related to the length of the lag phase.

3) Results of shaking culture tests indicated that addition of sea water provides buffer capacity to the system rather than accelerating nitrification by the effects of trace materials in sea water. In all cases of the run, degree of nitrification was preceded by the addition of sea water. As pH in the system has kept relatively high due to the buffur action of sea water, activities of Nitrobacter seemed to be restrained and nitrite nitrogen be readily to exist in the system.

4) Growth rate on Nitrosomonas in 40% mixture of sea water under the existence of Tris-buffer was about 1.34 times as much as that in 20% mixture. This was probably due to the favorable effect of Mg in sea water. However, ammonium oxidation rate per Nitrosomonas was more in 20% mixture than that in 40%. 5) In a system in which culture time was less that nitrifiers' generation time, phosphate was effective to accelerate nitrification. Optimum dose was 13mM for ammonium oxidation. 6) High dose of anions deteriorates the rate of nitrification under the existence of phosphate. Deterioration effect of anions is summarized as follows: NO_3^-(29mM)>F^-(48mM)>SO_4^{2-}(76mM)>I^-(80mM)>Br^-(140mM)>Cl^-(170mM). Numbers in parenthesis are the critical concentrations which deteriorate the rate of nitrification down to 50%.

Bacteria culture procedure: 1) Dispersion technique of activated sludge for bacteria determination applied here was ultrasonic shredding. Selected power for the apparatus was 25 W and 2.5 minutes for shredding. Those are recommended values. 2) Recommended procedures for bacteria determination for counting are 10 day incubation for heterotrophs in 20°C with Trypton media and for nitrifiers 20 – 40-day when IBP method is applied.

2. SUCCESSION OF MICROORGANISMS IN ACTIVATED SLUDGE IN NITRIFICATION

It is a well known fact that microorganisms contained in activated sludge differ in types and compositions with influent quality, organic loads, return sludge rate and dissolved oxygen and so on.

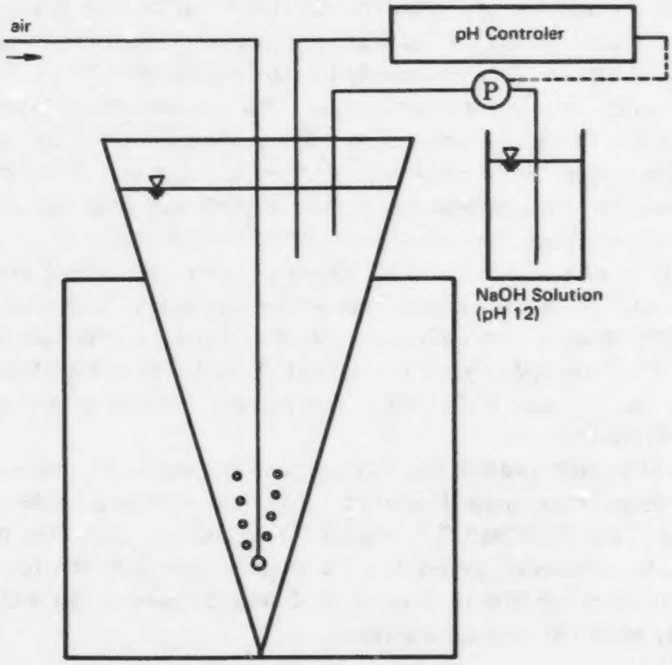
This particular study was conducted to determine operating conditions that would promote nitrification at activated sludge treatment facilities by conducting examinations of the changes of hetrotrophs, nitrification bacteria, protozoa and metazoa besides studying organic removal and nitrification rate of the mixed culture system based on SRT as an indicator of activated sludge. SRT is one of the factors that is detrimental in dominating the types and compositions of micro organisms and is an indicator for the generation time of microorganisms.

Studies were also made on the influence of sudden change of the pH against nitrification.

2.1 EXPERIMENT METHOD

Five Inhoff type cones were used for the experiment. (Fig. 2.1).

Fig.-2.1 Batch Type Testing Apparatus



The synthetic sewage mainly composed of protein such as meat extract, peptone was fed to the aerators by batch method with aeration time being 22 hours and sedimentation time 2 hours. Activated sludge obtained from domestic sewage treatment plant was seeded to the each aerator. The experiment was conducted in a 20°C constant temperature room. pH was controlled at a range between 7 and 8 continuously by automatic regulator, and the SRT was set by controlling MLSS in the aerators. Namely, influent organic load was kept constant while the MLSS was adjusted each aerator at 500, 1,000, 1,500, 3,000 and 6,000 mg/l.

Nutrient bacteria, nitrite bacteria, nitrate bacteria, protozoa and metazoa in mixed liquor were determined beside water quality analysis.

The trypton glucose culture medium was used for determination of heterotrophs, culturing at 20°C for 10 days. All colonies emerging from the culture were counted. The nitrification bacteria determination was followed by the IBP.

2.2 TEST RESULTS AND DISCUSSION

The summary of the results is shown in Table 1. The shortest SRT was about 4 days while the longest 60 days. As to the sludge characteristic, when the F/M ratio was high, SVI became 125 and growth of filamentous bacteria was observed resulting in bad sedimentation condition. However, when the F/M ratio maintained lower, or in the longer SRT condition, protozoa in the sludge became affluent. And the SVI was easy to be maintained less than 100.

The organic load was estimated to be between 0.2 and 1.5 g/ss.g/day from the following: TOC-SS load 0.06 – 0.58 g/ss.g/day; and BOD-SS load based on the relationship between TOC and BOD (observed ratio: BOD = 2.78 TOC).

Table-2.1 Average Values of Analytical Results

Phase	A	B	C	D	E
SRT (day)	3.65	5.82	9.60	18.01	59.3
MLSS (mg/l)	670	1,202	1,626	3,083	5,979
MLVSS (mg/l)	647	1,089	1,465	2,585	4,618
MLVSS/MLSS	0.966	0.906	0.901	0.839	0.772
TOC Load (mg/ss·g/day)	538	308	213	115	58.6
Removed TOC (mg/ss·day)	489	288	198	107	54.6
Removal Rate (%)	91.0	93.5	92.5	93.0	93.1
SS (mg/l)	49.1	32.4	31.4	39.2	22.2
Removal Rate K–N	43.6	50.8	59.4	92.1	93.6
(%) Org–N	91.9	95.4	91.3	91.2	91.5
T–N	35.1	25.7	20.4	3.8	–0.1
Nitrification Rate (%)	11.6	36.5	55.8	97.1	99.0

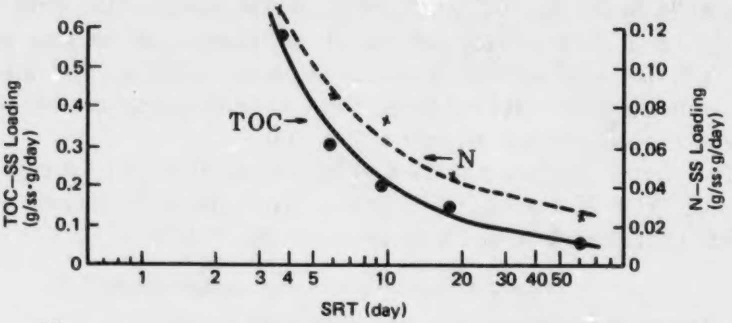
The SRT in this paper is defined as below.

$$\text{SRT (days)} = \frac{V \times S}{\Delta s + ss.q}$$

V: Aerator capacity (liter), S: MLSS (mg/l),
s: Excess activated sludge (mg), ss: effluent SS (mg),
q: flow (l/d).

SRT expresses the retention rate of activated sludge within the system. At normal condition, the SRT is subjected to sludge production which depends upon the volume of removed organic matters to be converted into sludge. The conversion rate to sludge differs according to sewage strength and for the municipal sewage the rate is about 0.7 while the synthetic sewage used in this test was about 0.21 which means the sewage is easily degradable. The relation between organic load and SRT in this study is shown in Fig. 2-2. The longer the SRT the smaller the F/M ratio.

Fig.-2.2 SRT vs Loading

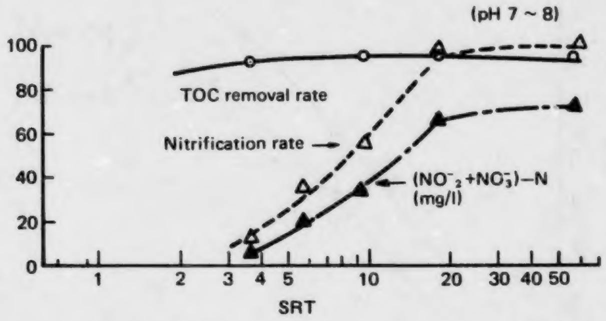


2.2.1 SRT and Removal Rate

The removal rate of organic substances under each run is in the range of 91 to 93.5 percent, and there was little difference in removal rate.

The removal rate of nitrogen in terms of TKN increased in proportion to SRT and reached 90% when SRT exceeded 18 days. In each run, organic nitrogen was removed mostly and removal rate was more than 91%. Total nitrogen removal rate was 35% at about four days of SRT. The removal rate decreased in proportion to the increase in SRT while nitrification rate increased. (Fig. 2.3). When SRT is kept over 18 days, mostly all nitrogen were converted to nitrate. When SRT increased, self oxidation in the sludge resulted in the release of ammonium to increase the total amount of nitrogen in effluent. The nitrate concentration, up until the 18 days of SRT, followed a straight ascending line. And beyond the 18 days, amount of TKN in the influent served as the limiting factor of nitrification. Since the amount of nitrite was extremely low in this SRT range, there was a possibility that the nitrification rate was hindered by nitrite in system.

Fig.-2.3 TOC Removal Rate, Nitrification Rate and SRT



2.2.2 Bacteria Population and SRT

The bacteria growth rate can be described as follows:

$$\frac{dN}{dt} = \mu N \qquad (1)$$

or

$$\ln \frac{N}{N_0} = \mu (t_2 - t_1) \qquad (2)$$

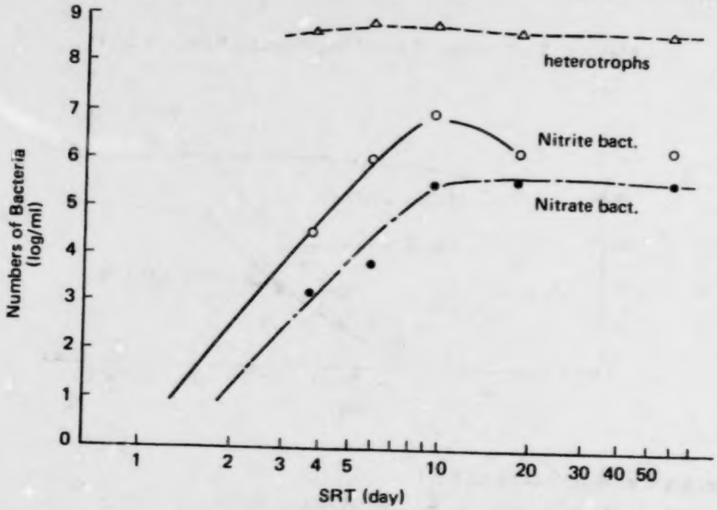
N_0: Number of bacteria at t_1
N: Number of bacteria at t_2
μ: Specific growth rate (l/t)
t_g: Average Generation Time (t)

$$t_g = \frac{\ell n 2}{u} = \frac{0.693}{u} \qquad (3)$$

Generally, the growth rate of nitrification bacteria is known to be between 10 and 12 hours, although some species require 20 or 40 hours. When compared to the growth of heterotrophs, the growth rates of autotrophs are slower owing to energy efficiency. The numbers of nitrification bacteria in activate sludge also changes in proportion to the length of SRT.

The relationship between bacteria number and SRT obtained is as shown in Fig. 2.4. With regards heterotrophs, constant number was maintained between 10^8 and 10^9 in this SRT range and thus the substrate in the system was regarded as the ruling factor. Furthermore, the value was close to the one obtained from the activated sludge at an existing plant. The nitrite bacteria increased in proportion to the SRT and reached the maximum value of 10^7 in number when SRT was 7 days while the nitrate bacteria were between 10^5 and 10^7. The existed number of nitrate bacteria was less than nitrite bacteria.

Fig.-2.4 Relation between SRT and Bacteria Number

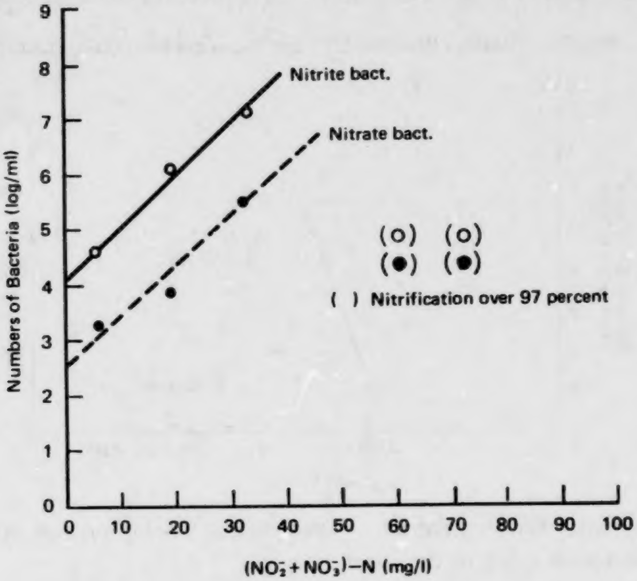


This fact also was confirmed by the water q[illegible]ty analysis that resulted in low concentration of NO^-_2 in effluent. This indicates that the NO^-_2 have acted as the ruling factor against the growth of nitrate bacteria.

When nitrification bacteria are to be maintained in maximum concentration with the substrates being the ruling factors similar to heterotrophs, the system requires SRT of over 10 days. At existing plants, although it is affected by operational factors, it is considered possible to designate operating SRT by estimating the maximum yield of nitrification bacteria by TKN concentration.

Since it is possible to consider that the maximum yield of nitrification bacteria is maintained at each designated SRT, from the relationship as shown in Fig. 2.4, the growth rate of nitrification bacteria (μ, 1/day) and generation time (tg.h) are calculated as follows: for nitrite bacteria $\mu = 2.26$, tg = 7.4h and for nitrate bacteria $\mu = 1.44$, tg = 11.6h respectively. Based on this generation time, from the relationship between nitrite bacteria and nitrate bacteria (Fig. 2-5), the minimum culture time or SRT can be obtained from Equation (2) (3) on bacteria multiplication rate after obtaining the nitrification bacterial number based on the assumption that the nitrification of TKN attained 100 percent.

Fig.-2.5 Nitrification Bacteria and Amount of Nitrification



2.2.3 Protozoa and Metazoa in the System

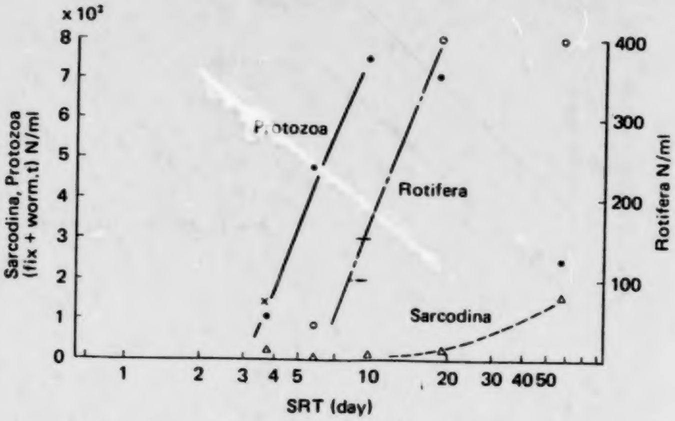
Table 2.2 shows the microscopic test results. From these results, the growth condition can be grasped to a certain degree. When organic load is high, the number of swim type microorganisms is great. And when the load decreases, the population of swim type diminishes accordingly. In this case, the Pleuromonas is quoted as the index. When settlerability of activated sludge sedimentation is good, it is generally regarded that the fix type and worm type microorganisms increase in number. And similar tendency was observed in this study. In the condition of aerobic digestion or extended aeration system, Rotifera, Arcella and Euglypha were frequently observed. In this case, they were observed when load was low. Also filamentous bacterium, related microorganisms to bulking, appeared in proportion to the load. The filamentous bacteria that appeared here included those in divided and shredded forms. Zoogloes also were detected as Z-ramigera and Z-filpendula coexisting.

The observation of the relationship of organic load and biological fauna indicated that the organic load and SRT are also in relationship and that the species of fast multiplying rate become dominant when SRT is short and those of slow growth rate appeared when SRT become longer to increase in variety. The relationship between SRT and biological fauna is shown in Fig. 2.6. Protozoa of fix type and worm type appeared when SRT was over 3 days, Rotifera over 6 days, and Sarcodina much longer. Similar to bacteria, the growth rate and generation time are calculated from Fig. 2.6 as follows: for Protozoa $\mu = 0.28$ 1/day, tg = 25 days; for

Rotifera $\mu = 0.20$ 1/day, tg = 35 days; and for Sarcodina $\mu = 0.05$ 1/day, tg = 13.6 days. Rotifera rank at the top of food chain and are polyphagia thus showing such tg level.

With regards loricate amoeba, it is often observed when organic loading is maintained low like in extended aeration or in aerobic digestion processes.

Fig.-2.6 Relation Between SRT and Protozoa Sarcodina, Protozoa



Being loricate, the percentage of such ameoba being preyed upon is low and their generation time being to the short category.

The microscopic studies of protozoa and metazoa revealed that the organisms that were identifiable also maintained intimate relationship with SRT in activated sludge. Furthermore, concerning the Flagellate organisms observed when organic loading was high, as the substrate became the ruling factor when organic substrate became low in concentration because they utilize soluble ingredients, they were handicapped in the race to consume energy. The swim type ciliata appeared were few when TOC-SS load were in the range of 0.2 – 0.3 mg/ss.g/day, and they were detected when the load was either higher or lower than this range.

2.2.4 Organism Index Predicting Nitrification Activity

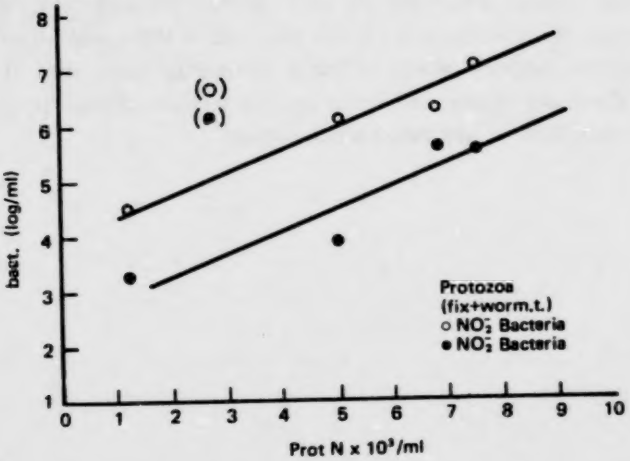
The attempt to grasp activated sludge condition by microscopic study results is one of the important procedures to maintain and control biological treatment.

To assume nitrification conditions from the results of microscopic studies, the relationship between nitrification bacteria concentration and protozoa (fix type plus worm type) shown in Fig. 2.4, Fig. 2.6 and Table 2.2 was used to develop Fig. 2.7. There is a clear corelation between nitrifier number and protozoa. It is possible to expect nitrification when protozoa of fix and worm type with comparatively long tg exist in quantity. This figure shows just an example, and it is necessary to accumulate data at actual existing plants. With regards Rotifera, Arcelia and Euglypha, they were often observed in existing treatment plants thus can be considered an effective index to predict nitrification activity.

Table-2.2 Organisms Survey Result

Phase	A	B	C	D	E
Sarcodina	243	29	29	200	1,622
Amoeba	243	29			
Arcella			29	171	371
Euglypha				29	1,257
Flagellate					
Pleuromonas	30,800	1,409	428	200	171
Ciliata					
Swim. type	1,267	0	324	1,200	1,000
Litonotus	1,170	0	324	1,200	1,000
Unknown g.	97				
Worm. type	0	340	1,228	1,114	486
Aspidisca		291	1,142	1,000	457
Euplates		49	96	114	29
Fix, type	1,215	4,672	6,285	5,877	1,771
Vorticella	389	1,321	571	620	257
Epistylis	729	1,268	1,714	571	200
Cpercuraria	97	437	829	143	200
Unknown g.		1,651	3,171	4,543	1,114
Rotifera	0	49	0	400	400
Filamentous bact. (%)	50	30	5	2	
SRT (day)	3.63	5.82	9.60	18.01	59.3

Fig.-2.7 Relationship between Nitrification Bacteria and Protozoa



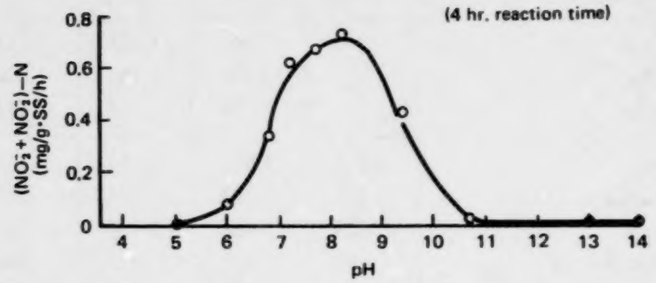
2.2.5 The Influence of pH on Nitrification Activity

Experiments were conducted to see the effect of pH change on the nitrification activity. Two procedures were chosen for the experiment. One is that four hour reaction time aerators were used for each pH condition between 4 and 14. The other is that after two hours of reaction at a set pH, the pH was adjusted to 7 for further reaction of four hours.

Concerning the optimum pH for nitrification bacterial activity, although researchers differ in opinion with regards the optimum pH range, it is generally regarded best between 7 and 8.5.

The reaction time for these experiments was too short for multiplication of nitrification bacteria, and the tests were to simply compare the pH effects against enzyme reaction. Fig. 2.8 shows the relationship between pH and nitrification rate.

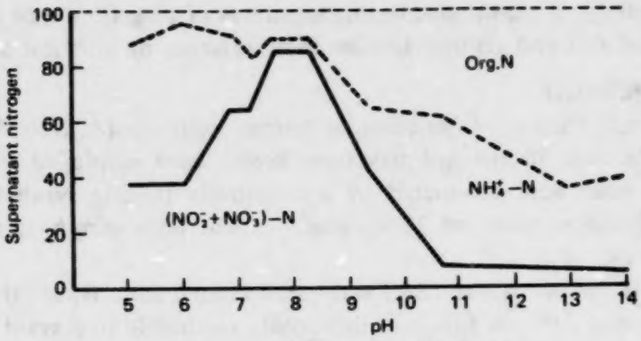
Fig.-2.8 Relation between pH and Nitrification



Maximum rate was obtained in the neighborhood of pH 8. It was 0.7 mg. $(NO_2^- + NO_3^-) - N$/g.ss/d. The Optimum pH range was 8 ± 0.5. The forms of pH and nitrogen were as shown in Fig. 2.9. On the acidic side, more ammonium residue was found, while on the alkaline side, more organic nitrogen was detected. In terms of total nitrogen amount, it was found increasing on the alkaline side perhaps due to elution of decomposing inert organisms and cells when pH was above 11.

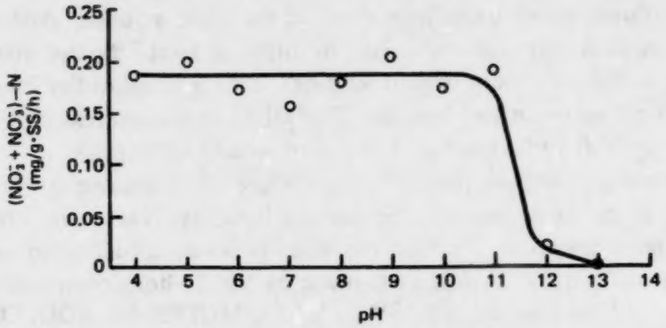
Fig. 2.10 shows that, the inhibition occurred when contacted with fixed pH for a short time and then adjusting the pH to 7. Within the range of ph 4 and 11, even when contact was made with each pH for two hours, little inhibition was observed against nitrification activity when adjusted to neutral later on. It also became obvious that there are strong resistance against sudden change in pH shock load deriving from the inflow of industrial waste water.

Fig.-2.9 pH and Form of Nitrogen



MLSS 2,400 ~ 3,000 mg/l

Fig.-2.10 Influence on Nitrification Rate when Adjusted to pH 7 after 2 Hours Contact with Set pH



MLSS 2,000 ~ 2,700 mg/l

2.3 SUMMARY

It became clear that the biological fauna in activated sludge was subjected to SRT. The amount of nitrification bacteria for sufficient nitrification was over 10^4 N/ml. The generation time of the biological fauna in mixed culture was as follows: Nitrite bacteria 7.4 hours; Nitrate bacteria 11.6 hours; Flagellate of fix and worm type 2.5 days; Rotifera 3.5 days and Loricate Ameoba 13.6 days. It is also effective to expect substantial nitrification by the control of treatment facilities with SRT required to maintain nitrification bacteria based on generation time. It is also possible to grasp the nitrification condition of plants by indexing Rotifera and Loricate Ameoba by microscopic examinations.

Maximum rate of nitrification in short time reaction was obtained at around pH 8. There were no changes observed in total nitrogen at pH 5 to 9, and the balance of nitrogen forms was affected by nitrification activity. As organic nitrogen increased on the alkali side, total nitrogen also increased. In the range of pH 4 to 11, even when contact was made with each pH for two hours, it was possible to continue nitrification activity after being neutralized.

3. NITRIFICATION STUDY BY CONTINUOUS OPERATION

In this study, a continuous testing apparatus of plug flow type aerators were used to control SRT and pH to determine their influence on nitrification.

3.1 TEST METHOD

In this test, four sets of continuous testing apparatus (A, B, C, D) of 60 liter aeration tanks and 30 liter sedimentation basins were employed. As shown in Fig. 3-1, the tests were conducted by continuously feeding synthetic sewage in constant temperature room of 20° Centigrade. The tests were broken up in three stages as follows:

I) The pH was set at 8 and SRT was varied in each unit. II) at pH 7, III) and at pH 6, Furthermore, the air flow was thoroughly controleld to prevent the DO from becoming a ruling factor in this experiment.

The pH control was done by Automatic pH Regulating Units at 2 points. The chemicals for pH control were NaOH and HCl diluted by tap water. SRT was indirectly controlled by introducing the following method. The concentration and flow rate of influent to all units were fixed at the same amount. And the MLSS concentration in each unit was maintained in different level. By this manner, each unit was fixed to have different organic loadings. SRT was controlled by extracting the set amount of waste activated sludge. The MLSS in the aeration tank of Unit A was set at 500 mg/1, B 1,000 mg/1, C 1,500 mg/l, and D 3,000 mg/l.

In the operation care was placed on controlling MLSS because it was necessary to maintain as constant as possible the organic loadings. Therefore, efforts were exerted to sludge withdrawal. Further, periodic checks were conducted on the flow rates of pumps. The daily sampling was made by the 24-hour composite sampling by analyzing the Transparency, pH, SV_{30}, MLSS, MLVSS, SS, BOD, COD, TKN, ammonia, nitrite and nitrate. A once weekly determination was conducted of hetrotrophs, nitrification bacteria, protozoa, and metazoa. The counting method based on plane culture was employed for hetrotrophs determination while the MPN Method was introduced to count nitrification bacteria after 100 days of culture. With regards counting of protozoa and metazoa, the microscopic method was incorporated. The culture media used for counting the bacteria were as follows:

Hetrotrophs: Trypton glucose culture medium (pH 7)
  Tripton: 20g, glucose 10g, and agar 9g are dissolved in one liter of water.
  Bacteria infection: 115°C, 30 minutes.
Nitrification Bacteria Quantitative Culture Medium:
  A Solution: KH_2PO_4 100 mg., $MgSO_4$ 50 mg, $CaCl_2$ 20 mg., $NaHCO_3$ 200 mg, EDTA-Fe 6 μg, $CaCO_3$, SiO_2 trace dissolved in one liter of ion exchange distilled water.
Nitrite Bacteria: A Solution plus $(NH_4)_2SO_4$ 30 mg
Nitrate Bacteria: A Solution plus KNO_3 30 mg
  (filtered by 0.2 μ membrane filter.)

Fig.-3.1 Outline of Testing Apparatus

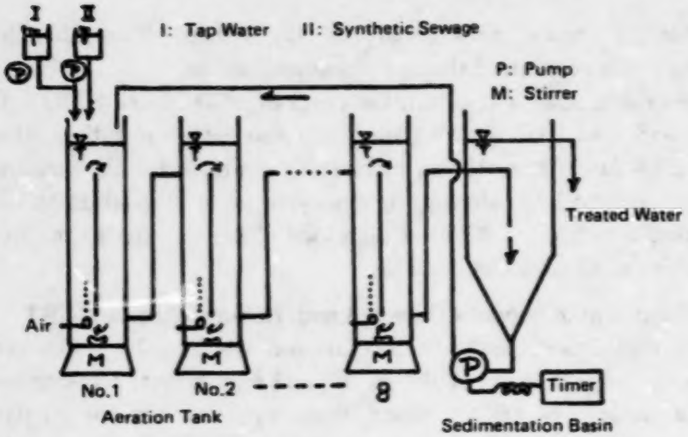


Table-3.1 Synthetic Sewage

(BOD = 150 mg/l)

Substrate	mg/1	Substrate	mg/1
Dextrin	23	NaCl	5
Meat extract	49	$MgSO_4$	3
Yeast extract	56	KH_2PO_4	14
Peptone	49	KCl	10

3.2 RESULTS AND DISCUSSION

Table 3.2 shows the summary of the results. The values represent each respective testing condition that are considered stable.

The organic load was within the range of BOD-SS Load 0.07 – 0.51 g/ss.g/day and COD-SS load 0.07 – 0.51 g/ss.g/day. And SRT was within the range between 2.7 and 22.4 days. The pH was successfully controlled in the range neighboring the set value. The SVI has shown a tendency to be in proportion to the organic load and marked a maximum 2,350 at high load. This was due to the growth of two or three species of filamentous bacteria.

3.2.1 Relationship between Organic Load, Removal Rate and SRT

With regards soluble (S) BOD, COD and T-N, Fig. 3.2 shows each relationship between the organic load per sludge unit and removal rate. The removal rate within this load range was: BOD, about 98 percent and virtually unaffected by load changes; and removal rate of total nitrogen increased in proportion to load-increase and remained virtually unaffected from pH. Similarly, Fig. 3.3 shows the relationship between loadings and the removed amount for each item. Each has shown that the removal rate per solid had the tendency to increase in proportion to the load applied loadings, and the gradients were: BOD, 0,984, COD, 0,853 and T-N, 0,447. These values express the removal rate per solid.

Fig.-3.2 Load and Removal Rate

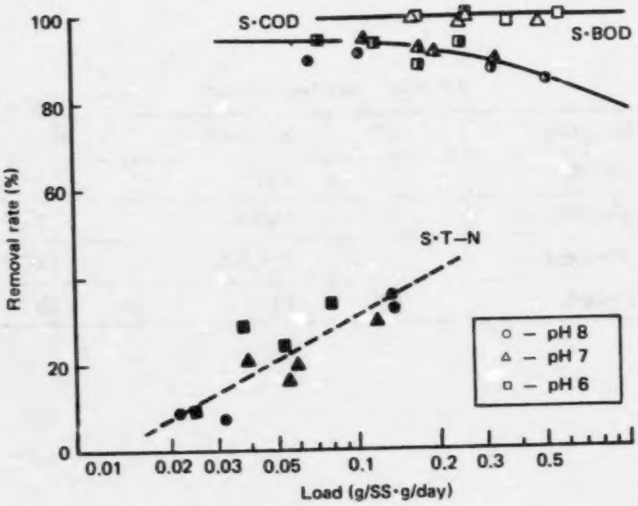


Fig.-3.3 Load and Removal Per Unit Sludge

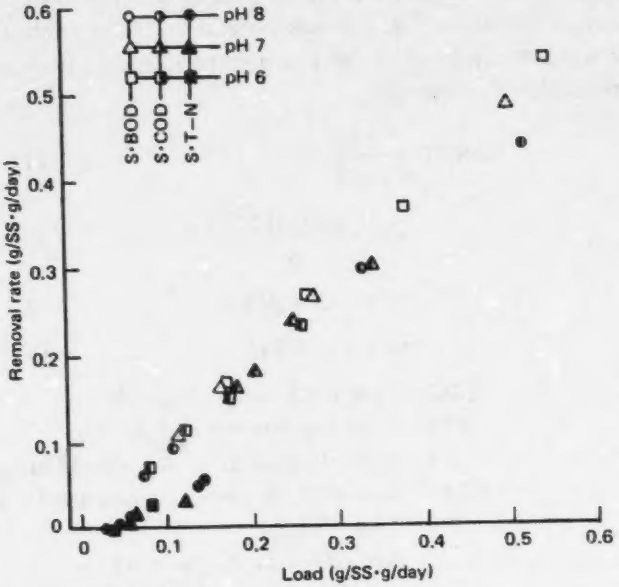
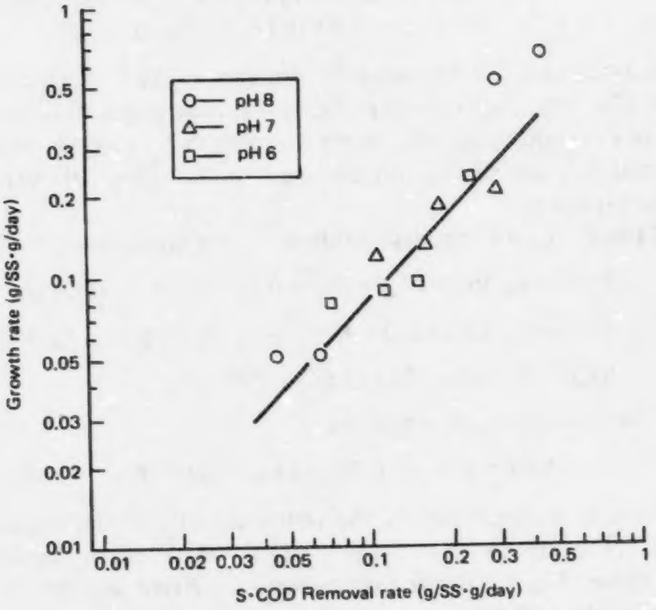


Fig.-3.4 Removal Rate and Multiplying Rate Per Unit Sludge



When examining the relationship between the removal rate of soluble COD per sludge unit and the amount of sludge production per solid, a positive primary interrelation as shown in Fig. 3.4 was obtained. If the relation between load and generating amount can be given, SRT to a certain degree can be predicted. The SRT can be expressed in the following:

$$\mathrm{SRT} = \frac{\Sigma \mathrm{SS}}{\Delta \mathrm{S}} \qquad (1)$$

$$= \frac{(1+\alpha)\,\mathrm{MLSS}.\,V_0}{\mathrm{S}} \qquad (2)$$

$$= (1+\alpha) \cdot F(R)^{-1} \qquad (3)$$

$$= (1+\alpha) \cdot F(L)_{\circ}^{-1} \qquad (4)$$

SRT: Sludge Retention Time (T)
ΣSS: Total Solid in the system (M)
ΔS: Multiplying Rate of Activated Sludge ($M.T^{-1}$)
MLSS: Activated Sludge Concentration in Aeration Tank (ML^{-3})
V_0: Aeration Tank Capacity (L^3)
α: Coefficient when Total Solid in the system is expressed in MLSS.
F(R), F(L): Multiplying Rate (Function of Amount per Solid (R) a..d Load (L)) (T^{-1})

Equation (1) is the fundamental equation of SRT which can be expressed in Equation (2). Also Equation (2) can be expressed as a function by the inverse number of the multiplying rate. When the relation between the multiplying rate and the removal rate per sludge unit and load can be given, SRT can be predicted by Equations (3) and (4).

F(R) and F(L) are obtained as follows by applying soluble COD.

$$F(R) = K_0.\mathrm{RCOD}\ C_0 + (K_0 = 0.864,\ C_0 = -0.002) \text{ and}$$

$$F(L) = K_0.K_1 \cdot \mathrm{LCOD} + K_0 C_1 + C_0\ (K_1 = 0.853,\ C_1 = 0.009)$$

$$\mathrm{SRT} = (1+\alpha) \cdot (0.737.\mathrm{LCOD} + 0.006)^{-1} \qquad (5)$$

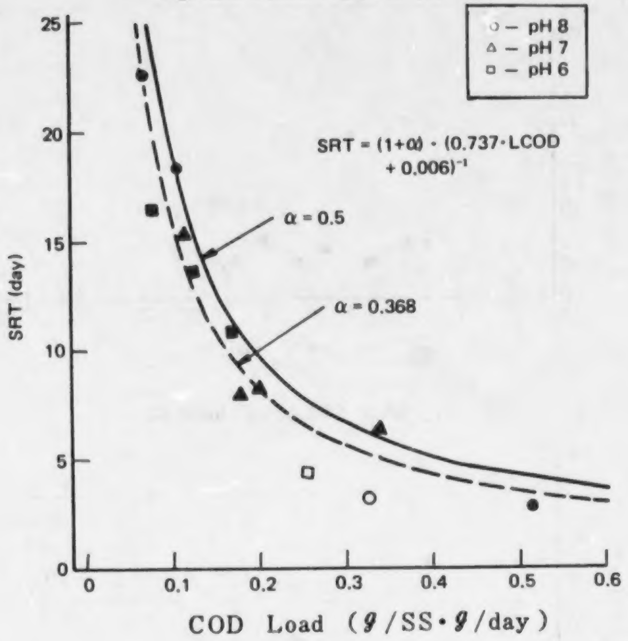
Thus the SRT was obtained as follows:

$$\mathrm{SRT} = (1+\alpha) \cdot (0.737 \cdot \mathrm{LCOD} + 0.006)^{-1} \qquad (5)$$

When α is approximated by the settler capacity of this testing apparatus, α is 0.5. Fig. 3.5 compares the results of SRT calculated by Equation (5) and the observed value. Fig. 3.5 shows that the values differed slightly. But if $\alpha = 0.368$ is used, it coincides with the observed values.

In this test, the effect of pH within the range of 6 – 8 was not clear on the relationship between organic load and SRT (Fig. 3.5). Perhaps, the pH range was so close to the optimum range of microorganisms like heterotrophs and protozoa in the activated sludge.

Fig.-3.5 COD-SS Load and SRT



3.2.2 SRT and Effluent Quality

Fig. 3.6.1, 2, 3 and 4 are the relationship between SRT and effluent quality in terms of BOD, COD, SS and Transparency. There is a tendency to deteriorate in effluent quality when SRT is either kept longer or shorter. As is seen in Fig. 3.6.1 and 2, the soluble BOD and COD were comparatively constant and low in any SRT. Effluent quality is influenced by effluent solids. Effluent solids increase by the bulking of filamentous bacteria when SRT is short, and increase of floatable solids due to denitrification in final settler or micronization of activated sludge when SRT was long. From this result, the optimum SRT range is to be between 5 and 10 days.

Fig.-3.6.1 SRT and Effluent BOD

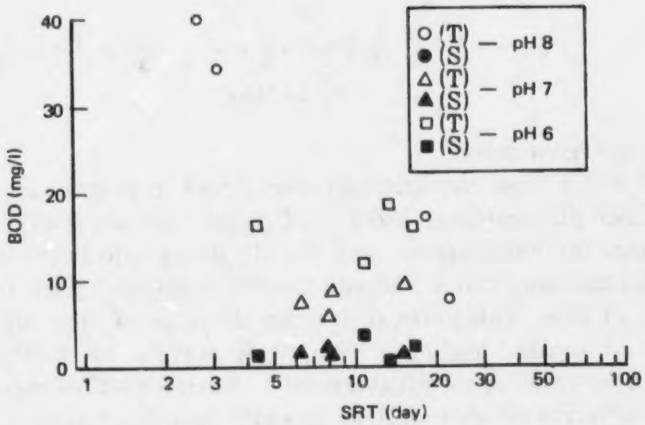


Fig.-3.6.2 SRT and Effluent COD

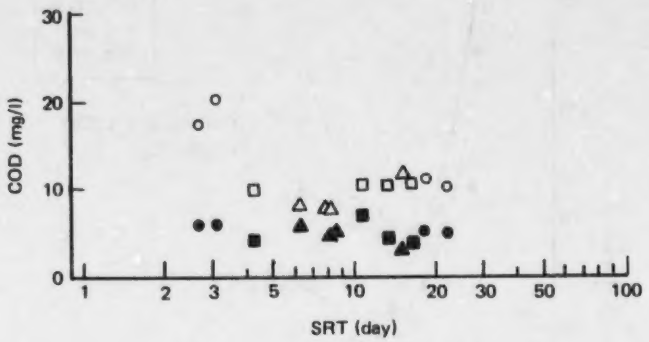


Fig.-3.6.3 SRT and Effluent SS

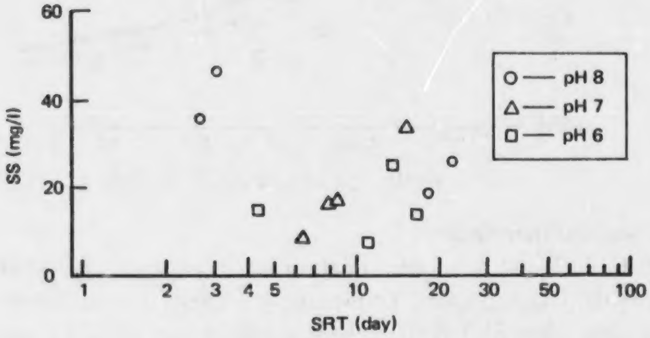
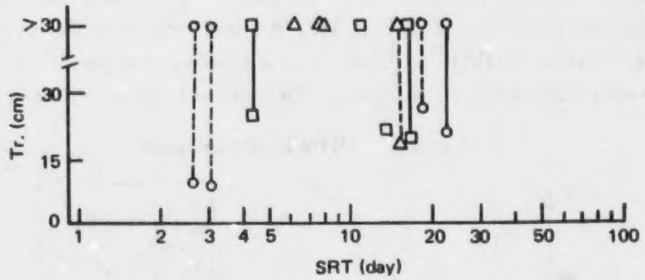


Fig.-3.6.4 SRT and Effluent Tr



3.2.3 SRT and Nitrification

Fig. 3.7 and 8 show the relation between SRT and nitrite and nitrate in the effluent, of each pH condition. From Fig. 3.7, the tendency is that the longer the SRT, the easier the nitrification. Also the pH greatly affects the whole process. Required SRT that nitrification start are; the SRT at pH 8 is 3 days; at pH 7, 6 days; and at pH 6, 11 days. This shows that within the range of these pH, when SRT is short or when organic loading is high, it is possible to maintain substantial nitrification by controlling the pH at around 8. When the SRT is maintained at over 15 days, the affects of pH are negligible, thus sufficient nitrification is expected.

Fig.-3.7 SRT and Nitrification

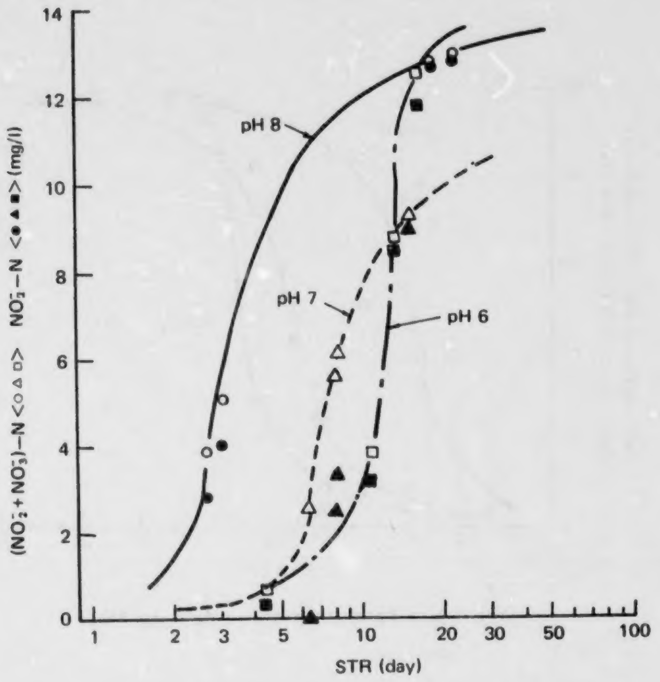


Fig. 3.8 shows the relation between SRT and nitrification rate. Of the nitrified nitrogen, the ratio of existing nitrite and nitrate is reported to change with the pH in the reaction tank. The condition allowing predominance of nitrate is pH 6 – 7 while at pH 8.0 – 8.6 there will be more nitrite existing. This explains that the activity of nitrite bacteria and that of nitrate bacteria change with pH. In this study, the existing ratio of nitrite of nitrified nitrogen was highest at pH 7 and at SRT 7 – 8 days, and at pH 8 the abovementioned pattern was not observed. In comparing the existing ratio of nitrite with SRT, when SRT was shrot or nitrification rate low, the existence rate was slightly high; and when the SRT was long the majority existed as nitrate. This tendency was also observed in the two other experiments. The multiplying rates of nitrite and nitrate bacteria from the two tests were 7 hours and 12 hours respectively. Accordingly, in a condition when nitrification rate was low or SRT short, as the existing amount of nitrite bacteria could not catch up with that of the nitrite bacteria, nitrite increased in amount. And when the nitrification rate was high or SRT long, the activities of both bacteria reached the same level allowing nitrified nitrogen to remain more in the form of nitrate.

Fig. 3.9 shows the relation between the pH and nitrification rate with SRT. When expecting more than 80 percent of nitrification at pH 6 – 7, the SRT should be more than 15 days. And it shows that in the neighborhood of pH 8, a reasonably sufficient nitrification can be expected by setting SRT at around 6 or 7 days.

Fig.-3.8 SRT and Nitrification Rate

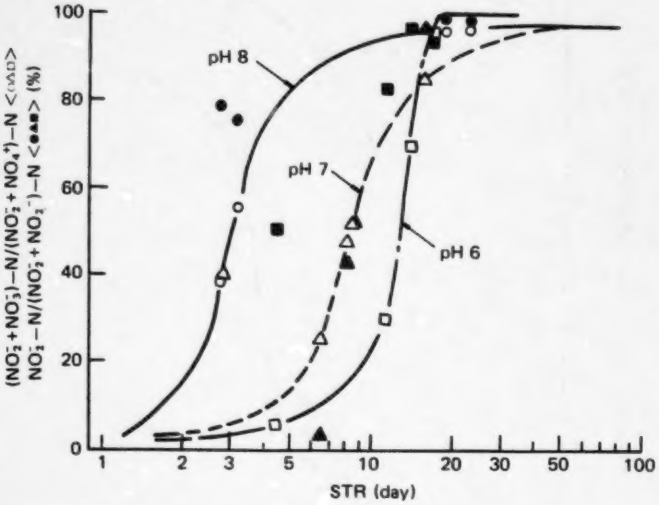


Fig.-3.9 pH and Nitrification Rate and SRT

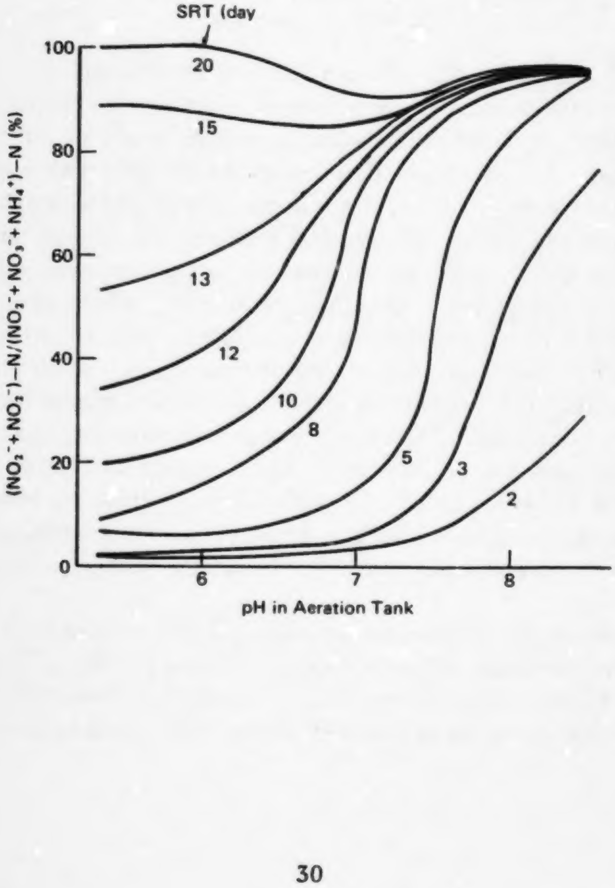


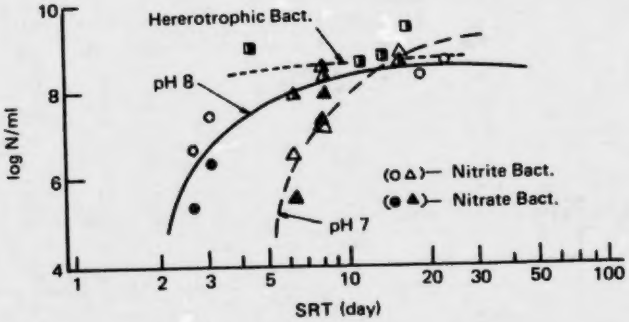
Table-3.2 Average Water Quality

		I pH 8				II pH 7				III pH 6			
		A	B	C	D	A	B	C	D	A	B	C	D
SRT (day)		2.68	3.06	18.27	22.4	6.31	8.18	7.96	15.25	4.33	10.86	13.47	16.43
ΔS (g/day)		14.04	17.13	7.74	8.96	8.42	11.13	9.32	13.09	12.81	7.41	10.35	12.91
pH		7.81 7.65 ~ 8.15	7.67 7.44 ~ 7.93	7.91 7.80 ~ 8.02	8.19 7.94 ~ 8.31	7.33 7.10 ~ 7.52	7.20 6.95 ~ 7.34	7.06 6.95 ~ 7.20	7.00 6.81 ~ 7.12	6.10 5.63 ~ 6.99	6.38 5.97 ~ 6.84	6.13 5.84 ~ 6.47	6.11 5.72 ~ 6.57
SV$_{30}$ (%)		85.4	85.0	70.7	45.0	72.0	78.3	91.8	93.5	19.6	28.3	20.5	41.8
MLSS (mg/l)		366	565	1,967	2,920	688	1,031	1,205	1,831	939	1,307	1,954	2,730
MLVSS (mg/l)		324	508	1,751	2,540	635	926	1,079	1,662	831	1,194	1,790	2,500
$\frac{\text{MLVSS}}{\text{MLSS}}$ (%)		88.5	86.7	89.0	87.2	91.6	89.8	89.6	91.1	87.6	91.7	91.7	93.1
SVI		2,350	1,770	345	146	1,123	791	791	542	209	217	105	146
BOD (mg/l)						70.7	87.8	71.3	71.0	116.4	117.8	126.3	111.7
COD (mg/l)		48.4	48.9	51.2	49.9	49.1	47.4	49.3	49.4	54.5	53.5	36.6	51.3
K–N (mg/l)		15.42	15.57	16.29	15.90	15.80	14.90	16.10	15.54	17.33	16.79	17.80	16.40
KH$_4^-$–N (mg/l)		9.41	9.41	9.07	8.81	10.20	9.84	10.27	10.24	4.38	4.52	5.09	4.00
Tr (cm)		(9.30) ~ 30 < $^{1/2}$	(8.81) ~ 30 < $^{3/7}$	(26.0) ~ 30 < $^{3/7}$	20.5 ~ 30 < $^{1/9}$	30 <	30 <	30 <	(17.6) ~ 30 < $^{3/5}$	24.5 ~ 30 < $^{1/5}$	30 <	20.8	19.4 ~ 30 < $^{1/8}$
SS (mg/l)		35.7	46.2	18.7	25.1	8.3	16.8	16.0	33.6	14.7	7.1	24.9	13.6
BOD (mg/l)	Total	39.9	34.2	17.13	7.70	7.18	8.42	5.68	9.17	16.13	11.78	18.48	15.87
	Soluble					1.62	1.00	2.03	1.34	1.21	3.56	0.52	2.01
COD (mg/l)	Total	17.19	20.0	10.83	9.85	7.87	7.43	7.41	11.10	9.81	10.33	10.03	10.36
	Soluble	5.81	5.81	4.98	4.58	5.60	4.39	4.22	2.86	3.93	6.71	4.02	3.63
K–N (mg/l)	Total	7.97	10.48	3.10	3.17	9.36	6.75	8.49	3.45	12.58	10.04	6.27	3.87
	Soluble	6.55	5.30	1.10	1.10	8.07	6.10	7.11	7.60	10.74	8.96	3.92	2.18
NH$_4^-$–N (mg/l)		6.25	5.56	0.31	0.51	7.77	5.47	6.38	1.61	10.73	8.96	3.88	0.53
NO$_2^-$–N (mg/l)		1.05	1.02	0.06	0.16	2.58	2.77	3.13	0.30	0.30	0.64	0.29	0.75
NO$_3^-$–N (mg/l)		2.82	4.04	12.73	12.75	0.01	3.32	2.45	8.94	0.32	3.17	8.47	11.79
BOD						0.496	0.270	0.247	0.1668	0.540	0.378	0.267	0.1696
COD		0.513	0.327	0.1065	0.0705	0.338	0.1996	0.1775	0.1147	0.252	0.1720	0.1214	0.0776
T–N		0.1376	0.1343	0.0320	0.0218	0.1180	0.0557	0.0597	0.0386	0.0801	0.0538	0.0375	0.0249
BOD	Total					89.8	87.0	96.6	87.1	85.9	90.3	85.6	85.6
	Soluble					97.6	98.5	97.1	98.1	98.9	97.1	99.6	98.1
COD	Total	52.3	40.5	76.0	78.8	83.8	83.4	84.7	76.2	81.8	80.5	82.2	79.4
	Soluble	84.9	86.5	90.7	89.0	88.5	90.5	91.0	93.9	92.7	87.4	92.9	92.8
T–N	Total	27.6	4.82	9.96	–5.57	17.60	11.01	10.88	1.13	22.4	17.36	13.18	–0.92
	Soluble	32.4	34.7	6.99	7.43	29.2	16.01	19.69	20.7	33.3	23.9	28.6	9.49
BOD	Total					0.446	0.235	0.225	0.1451	0.466	0.340	0.228	0.1380
	Soluble					0.484	0.266	0.239	0.1636	0.535	0.367	0.266	0.1664
COD	Total	0.293	0.1596	0.0843	0.0566	0.284	0.1680	0.1518	0.0855	0.207	0.1396	0.0997	0.0614
	Soluble	0.436	0.297	0.0963	0.0641	0 300	0.1815	0.1637	0.1076	0.234	0.1508	0.1127	0.0720
T–N	Total	0.0501	–261 × 10^{-3}	4.46 × 10^{-3}	–0.888 × 10^{-3}	27.4 × 10^{-3}	6.25 × 10^{-3}	10.93 × 10^{-3}	–0.949 × 10^{-3}	19.88 × 10^{-3}	9.59 × 10^{-3}	4.92 × 10^{-3}	–0.633 × 10^{-3}
	Soluble	0.0574	0.0508	2.30 × 10^{-3}	2.01 × 10^{-3}	35.7 × 10^{-3}	8.79 × 10^{-3}	17.57 × 10^{-3}	8.23 × 10^{-3}	28.2 × 10^{-3}	12.89 × 10^{-3}	10.88 × 10^{-3}	2.14 × 10^{-3}

Table-3.2 (continued)

		I pH 8				II pH 7				III pH 6			
		A	B	C	D	A	B	C	D	A	B	C	D
Nitrification rate (%)	$\frac{NO_2^- - N}{(NO_2^- + NO_3^- + NH_4^+) - N}$	9.97	18.25	0.75	0.99	20.1	24.3	25.5	2.80	2.73	5.20	2.40	5.80
	$\frac{NO_3^- - N}{(NO_2^- + NO_3^- + NH_4^+) - N}$	28.2	37.1	95.6	95.3	5.04	27.2	22.1	82.2	2.88	24.7	67.2	90.3
	$\frac{(NO_2^- + NO_3^-) - N}{(NO_2^- + NO_3^- + NH_4^+) - N}$	38.2	55.4	96.3	96.3	25.1	51.5	47.6	85.0	5.61	29.9	69.6	96.1
	$\frac{NO_3 - N}{(NO_2^- + NO_3^-) - N}$	78.6	75.4	99.2	98.7	0.31	51.2	42.6	96.7	50.3	82.6	96.6	94.0
Income and Outgo (%)	$NO_2^- - N$	7.87	6.24	0.38	1.01	16.19	15.1	15.88	1.76	1.83	3.92	1.72	4.58
	$NO_3^- - N$	17.49	26.7	74.2	82.4	0.08	21.9	18.93	62.9	1.88	18.83	47.7	72.5
	$NH_4^+ - N$	38.9	31.8	1.79	3.16	48.5	41.1	38.7	7.61	62.9	53.4	21.7	3.29
	Soluble Org-N	3.13	0.40	6.61	3.79	7.55	5.80	6.87	7.08	0.08	0.00	0.29	10.15
	Total Org-N	7.88	24.9	15.19	16.72	19.09	10.80	15.67	26.6	10.98	6.51	14.42	20.6
Nitrite Bacteria (/ml)		4.9×10^6	2.8×10^7	2.4×10^8	4.9×10^8	3.3×10^6	1.7×10^7	1.7×10^7	7.0×10^8				
Nitrite Bacteria (/ml)		2.4×10^5	2.4×10^6	2.4×10^6	2.4×10^5	3.3×10^6	7.9×10^7	7.2×10^7					
Heterotrophic Bacteria (/ml)						7.6×10^7	2.4×10^8	3.2×10^8	5.3×10^8	1.0×10^8	4.3×10^8	6.4×10^8	2.6×10^8

Fig.-3.10 SRT and Bacteria Population



3.2.4 SRT and Microorganisms

Fig. 3.10 shows the relation between SRT and the numbers of heterotrophs and nitrification bacteria. The number of heterotrophs was somewhere around 10^8 N/ml, and no drastic change was observed by change in SRT. This value was close to the one in the activated sludge in the actual sewage treatment plant. The number of nitrification bacteria amounted to between 10^5 and 10^8 per milliliter and it was subjected to change to both SRT and pH in the aeration tank. It is considered that this is due to the difference in the activities of nitrification bacteria as against the organic load and pH. Considering the fact that the optimum pH of nitrification bacteria is in the neighborhood of 8 when high, they become formidable against inhibition caused by organic load, leading to maintenance of higher activity and multiplying rate than other pH. When comparing nitrate and nitrite bacteria from Fig. 3.10, at low SRT range, the existing patterns of both bacteria closely resembled with each other. In this test, as the influent did not contain any nitrite nitrogen, the existing quantity of nitrite bacteria or the amount of nitrite was considered to have accelerated the growth of nitrate bacteria.

Fig.-3.11.1 SRT and Metazoa

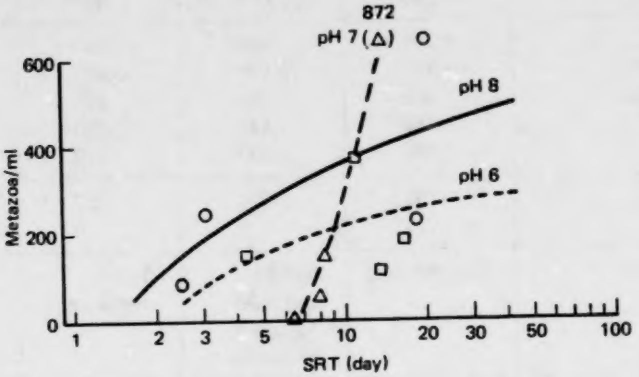


Fig.-3.11.2 SRT and Amoeba

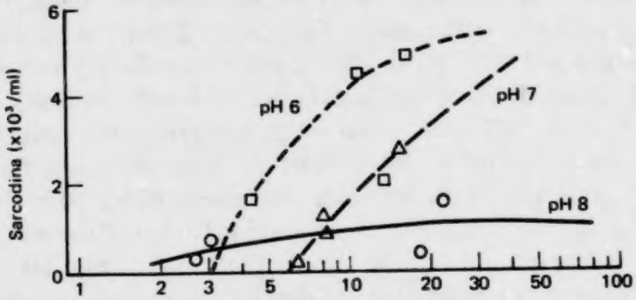


Fig.-3.11.3 SRT and Amoeba

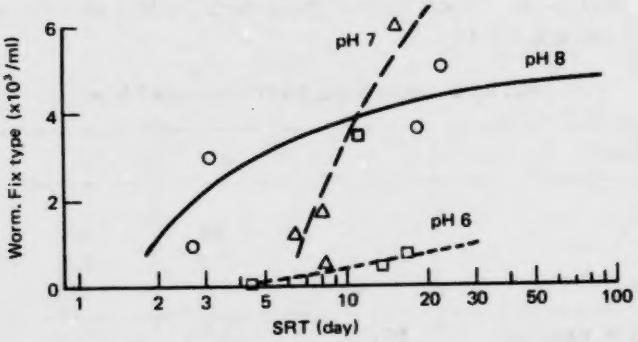


Table-3.3.1 Microscopic Test Results (pH 8 N/ml)

Phase	A	B	C	D
Sarcodina	240	657	400	1,510
Flagellata	4,305	3,430	147	783
Ciliata	1,146	3,199	3,906	6,270
Swim. type	200	226	306	1,266
Worm. t.	360	1,143	2,680	3,744
Fix. t.	586	1,830	920	1,260
Rotifera	67	244	227	444
Nematoda	27	0	0	167
	Pleuromonas	Arcella Euglypha Aspidisca Vorticella	Arcella Aspidisca	Euglypha Aspidisca
SRT (d)	27	3.1	18.3	22.4

By classifying the biological fauna in the activated sludge into Metazoa, Rhizapoda and activated sludge ciliata, Fig. 3.11.1, 2 and 3 show the relationship between each group and SRT. Table 3.3.1, 2 and 3 show the population of the main organisms that appeared. Each biological fauna has shown a tendency to increase in number on the high SRT side. Also when comparing this with pH, Rotifera increased in number at pH 7. In contrast to other biological fauna, Rotifera remained formidable against pH influence and proved highly adaptable to pH. In this pH range, it was amoeba galore in the lower pH area and showed a tendency to decrease in the higher pH zone. In this study Englypha, Arcella and other loricate amoeba observed in extended aeration or long SRT system were found in abundant. Loricate amoeba is one of Protozoas upon which Rotifera find them difficult to prey and this is why they still exist in this low pH 6.

The activated sludge ciliata, on the otehr hand, unlike amoeba, were found in less quantity in the lower pH range.

Table-3.3.2 Microscopic Test Results (pH 7 N/ml)

Phase	A	B	C	D
Sarcodina				
Euglypha	50	286	457	1,357
Arcella	86	407	422	793
Amoeba	43	157	343	607
Flagellata (Pleuromonas)	50,840	22,293	2,382	893
Ciliata				
Swim. type	436	3,607	3,979	750
Litonotus	129	2,400	514	43
	307	1,207	3,465	707

Table-3.3.2 Microscopic Test Results (pH 7 N/ml) (continued)

Phase	A	B	C	D
Worm. type	222	244	1,365	5,579
Aspidisca	214	222	1,179	4,300
	8	22	186	1,279
Fix. type	916	257	300	386
Vorticella	522	50	43	86
Zoothamnium	300			
Carchesium	72	107	43	
Tokophrya		14	50	
	22	86	164	300
Rotifera	0	108	43	872
Nematoda	0	36	8	0
SRT (d)	6.3	8.2	8.0	15.3

Table-3.3.3 Microscopic Test Results (pH 6 N/ml)

Phase	A	B	C	D
Sarcodina				
Euglypha	930	3,180	1,100	2,960
Arcella	0	30	190	240
Amoeba	550	900	720	1,570
	150	380	30	120
Flagellata (Pleuramonas)	21,780	3,060	10,870	3,050
Ciliata				
Swim. type	10	0	300	200
Litonotus	10	0	100	50
			200	50
Worm. type	40	0	340	100
Aspidisca	40	0	340	100
Fix. type	20	3,460	130	630
Vorticella	20	1,960	70	200
Epistylis	0	1,350	0	100
Tokophrya		50	20	10
		100	40	320
Rotifera	150	370	110	180
Nematoda				
SRT (d)	4.3	10.9	13.5	16.4

3.3 SUMMARY

(1) Load and SRT.

The relation between load and SRT was proved to be described by Equation (5). It is recommended that the coefficient α be the ratio of the settler capacity to the aerator capacity.

(2) SRT and Nitrification.

The study results revealed that at pH 8, nitrification can be expected in only half of SRT than at pH 7. When applying this to existing conventional treatment plant operation, either the load should be kept low to maintain long SRT or the pH in the aeration tank should be kept high. The application of the former to the existing treatment methods is difficult because capacity is limited at the most plants. Also the employment of the latter requires installation of the pH adjustment facilities.

(3) SRT and Microorganisms.

The number of Heterotrophs was 10^8 per milliliter, and the amount remained unaffected by SRT changes. The number of nitrification bacteria was between 10^5 and 10^8 per milliliter and was strongly affected by the length of SRT and also by pH. When SRT was fixed, the amount of nitrification bacteria increased in the order of pH 8, 7 and 6.

In this study, on the slightly alkaline side of the pH, protozoa and activated sludge ciliata were detected in galore, while on the acidic side more amoeba were found. On the other hand, under long SRT condition, both had shown a tendency of increasing. Rogifera and Sarcodina are regarded as useful index organisms of nitrification condition.

(4) SRT and Effluent Quality.

The optimum SRT was 5 – 10 days. When SRT was kept short, it induced a bulking condition. When SRT was long, outflow of minute floc and floating of sludge from settlers perhaps due to denitrification occurred, at the final sedimentation basins.

4. EXPERIMENTS CONCERNING EFFECTS OF TRACE ELEMENTS ON NITRIFICATION

There are two procedures to determine effects of trace elements on nitrification acceleration.

1) Short terms culture method; to determine effects of trace element addition on nitrification rate within a short period (4 – 5 hrs) in which growth of nitrifiers is negligible. Namely, this method is to determine the effect of trace element addition on the enzyme reaction.

2) Long term culture method; to determine effects of trace elements on growth of nitrifiers and nitrification activity of nitrifiers by measuring amount of nitrifiers and nitrified nitrogen in effluent.

In this study, long term contact method has been adopted.

4.1 TEST METHOD

The activated sludge at sewage treatment plants, (chlorine concentration of 40 – 50 mg/l as Cl) where substantial nitrification achieved was used as nitrification bacteria. Ultrasonic treatment (50Hz, 25 watt 5 ~ 10 minute) was conducted to kill protozoa to prevent them to prey on nitrifiers. The sludge was washed two or three times with ion exchange distilled water to release soluble metals.

To alleviate the influence of buffer solution and sea water, the analysis was conducted by diluting samples of 10 to 20 times.

Test 1) The Effects of Sea Water under the Existence of Tris Buffer

Effect of sea water on nitrification was tested adding sea water of 0, 20, 40, 60, 80, and 100% to the influent.

As buffer solution, 50mM Tris buffer (tris hydroxymethyl aminomethane) was used as it was considered comparatively low in physiological influence. $(NH_4)_2SO_4$ and $NaNO_2$ were added at the concentration of 30 mg.N/l as sources of ammonia and nitrite respectively. Under the existence of 50m/M Tris-buffer, activated sludge collected at sewage treatment plant was used after one week of aeration. The sea water was prepared according to the Lyman and Fleming Synthetic Seawater Composition List.

In 3-liter vessels, sea water, Tris-buffer, washed activated sludge (MLSS adjusted to 520 mg/l) and substrate were added in that order to total 3 liters each and to reach the designated concentrations respectively. Then the mixtures were separated in 500 ml flasks by 100 ml transfer pipers, and the flasks were attached to shaking culture units. The shaking culture was conducted at an amplitude of 50 mm, and shaking frequency of 10 times per minute.

Test 2) Effects of Phosphate Buffer

From the results of Test 1) Tris-buffer, that inhibit nitrification bacteria, was found inappropriate for use in the nitrification tests. To examine the effects of phosphate ($Na_2HPO_4 + NaH_2PO_4$) which is the alterative buffer solution against nitrification, the short incubation method was introduced.

To reach the designated concentrations, one-liter cylinders were filled with phosphate (When NH_4^+ substrate: 10^{-4}, 5×10^{-4}, 10^{-3}, 10^{-2}, 25×10^{-2}, 5×10^{-2}, 10^{-1} M phosphate. When NO_2^- substrate: 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}, 10^{-1} M phosphate), 2000 mgN/l $(NH_4)_2SO_4$ or $NaNO_2^-$ . . . 25 ml, and washed activated sludge collected at sewage treatment plant (When NH_4^+ substrate) 21800 mg/l . . . 100 ml or activated sludge (When NO_2^- substrate) 24800 mg/l . . . 150 ml in that order to total 1 liter each. Then the mixtures were aerated and the pH adjusted between 7.5 and 8.0 of NaOH. And finally the amount of nitrite bacteria contained in 0, 0.5, 5, 10 and 20mM solutions of NH substrate 24 hours later were measured.

Test 3) The Effect of Mg^{2+} on Nitrification under the Existence of Phosphate

Phosphate of 2mM was added to the solution of $MgSO_4$ (002, 0.2, 2, 50mM), $MgCl_2$ · · · 50mM, and $(MgCO_3)_4$ $Mg(OH)_2$ . . . 0.04mM (Mg = 0.2mM) to determine the effects of Mg in short period incubation.

In 1-liter cylinder, 40 ml of 0.5M Phosphate was added. Then Mg compounds were added to reach the designated concentration. This was followed by the annexing of 100 ml of washed activated sludge (MLSS 19400 m/l) and 25 ml of $(NH_4)_2SO_4$ 2000 (mg/l as N). The mixtures were aerated and five minutes later amount of ordinary bacteria were measured.

Test 4) Effect of Anion under the Existence of Phosphate

Under the existence of phosphate, NaF, NaCl, NaBr, NaI, Na_2SO_4 and $NaNO_2$ of 10^{-4}M, 10^{-3}M, 10^{-2}M and 10^{-1}M were added respectively to examine the affects of anion against nitrification.

4.2 TEST RESULTS

Prior to the buffer addition study, tests to determine the effects of sea water on nitrification was conducted.

The results are shown in Figs. 4.1, 4.2 to 4.4. Since these tests were conducted by adjusting the pH only once every day, when the sea water addition rate was low or the buffer capacity low, the pH also decreased, making it impossible to maintain optimum pH. The more the pH was lower than the optimum pH, the apparent nitrification rate decreased. In short, it is assumed that the actual nitrification rate will increase to the extent of the concentration of sea water as shown in Figs. 4.1 and 4.2: This indicated that the optimum pH must be maintained in this manner to make comparisons of nitrification rates. Then, the experiments were extended to be done under the addition of buffer solution to remove the effect of pH change.

Fig.-4.1 Effect of Sea Water on Nitrification Rate (fresh water nitrifiers CL = 50 mg/l)

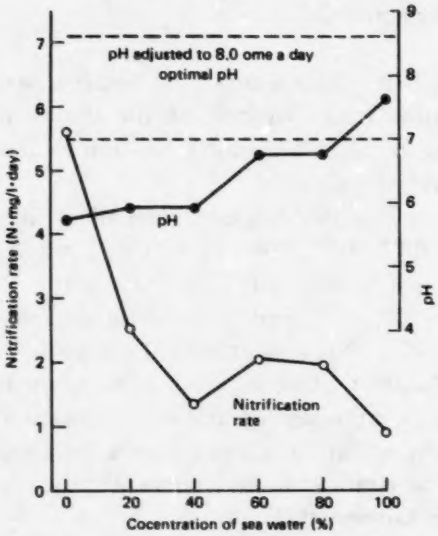


Fig.-4.2 Effect of Sea Water on Nitrification Rate (Activated Sludge of 6000 – 7000 CL plant)

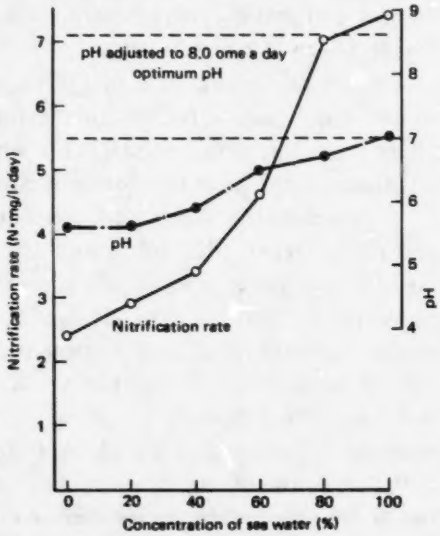


Fig.-4.3 pH and Nitrification

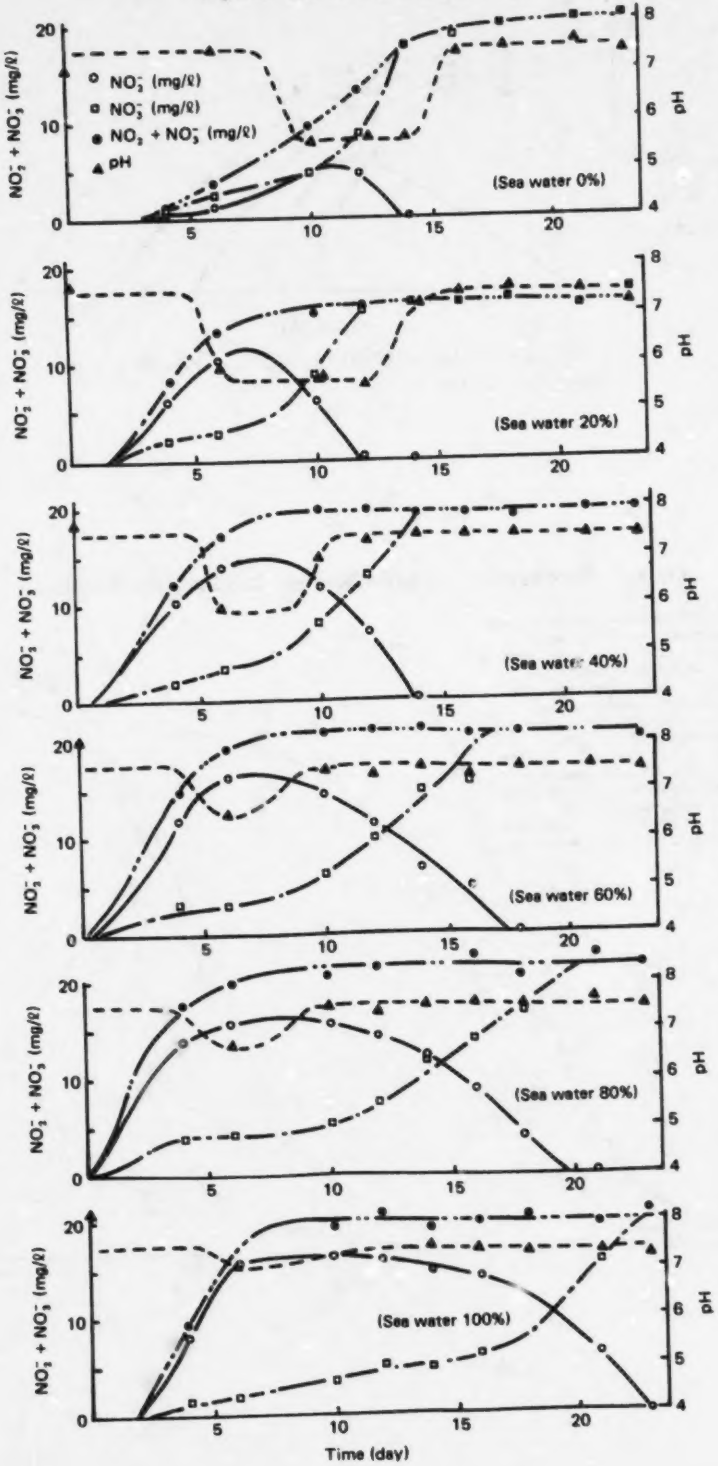
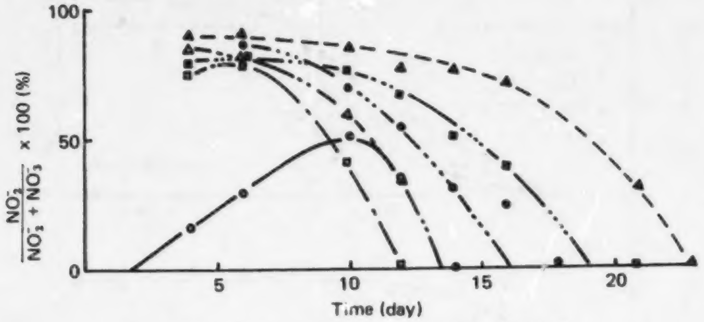
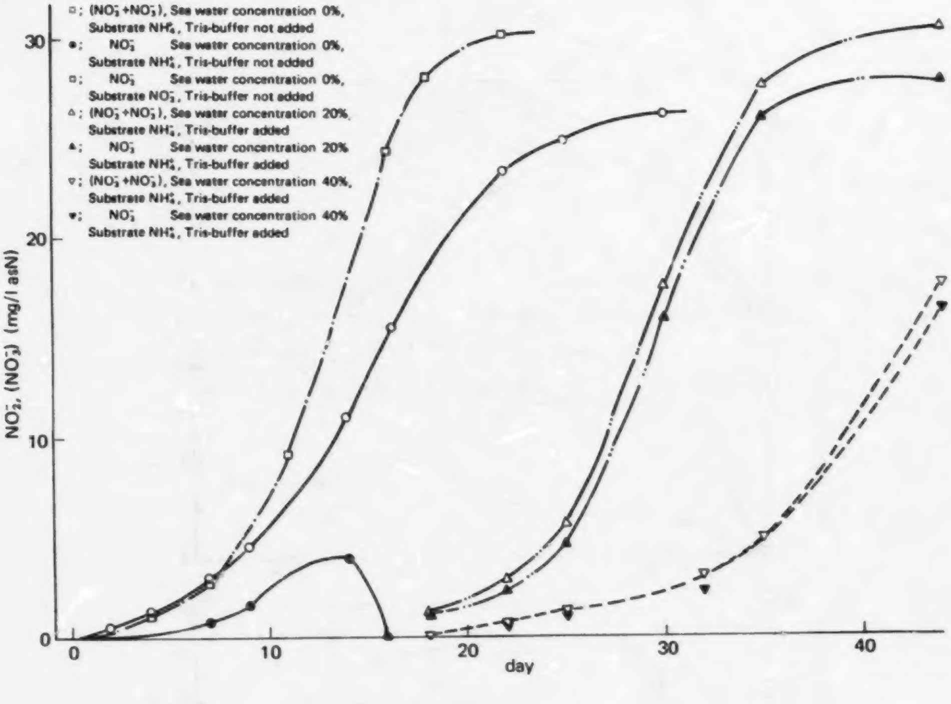


Fig.-4.4 Sea Water vs. Percentage of Nitrite



○ Sea water: 0 (%), □: 20 (%), △: 40 (%), ●: 60 (%),
■: 80 (%), ▲: 100 (%)

Fig.-4.5 Nitrification in Different Substrat. and Sea Water Conditions



Test 1) Influence of Sea Water under the Existence of Tris Buffer

Figure 4.5 shows the relation between nitrified nitrogen (NO^-_2, $NO^-_2 + NO^-_3$) in effluents and reaction time (days), under the different sea water concentration, fed forms of substrate nitrogen (NH^+_4 or NO^-_2).

Similar to the result shown in Fig. 4.1, the inclination was that the nitrification declined when sea water concentration increased. The rates were 2.38 (mg/l.day) in 20 percent sea water concentration and 1.21 (mg/l.day) in 40 percent sea water concentration. These values relate that the nitrification rate in 40 percent sea water was only about 1/2 of that of 20 percent, indicating the fact of sea water inhibition.

Inversely, the growth rate of nitrite bacteria as shown in Fig. 4.6 was higher in 40 percent than in 20 percent sea water with the values being 0.126 (generation number) and 0.0941 respectively. This means that the growth rate in 40 percent sea water solution was about 1.34 times faster than in 20 percent. The generation time, on the other hand, was 2.4 days in 20 percent sea water and 5.5 days in 40 percent.

Test 2) Effect of Phosphate Buffer

The ratios of nitrification rate of phosphate addition and without addition under the nitrogen substrate of NH^+_4 and NO^-_2 are shown in Fig. 4.7. It shows that the phosphate added as pH buffer had accelerated ammonia oxidation with the optimum concentration being about 13mM. The rate at this point was about twice the rate of that phosphate was not added (Control). However, when phosphate concentration was below 0.1mM, there was no action observed to accelerate ammonia oxidation. Also over 13 mM in concentration, the rate suddenly dropped. And when 100mM, the rate dropped about 45 percent against Control. The nitrite bacteria count in 0 – 10mM concentration, 24 hours later, roughly maintained the same population of 1.5×10^8 N/ml. However, the count in 25mM concentration dropped to about one third of the original level and was 4.9×10^7 N/ml.

Test 3) Effect of Mg on Nitrification under the Existence of Phosphate

To see the effect of Mg^{+2} addition, ratios of nitrification rate of Mg^{+2} level against control rate are shown in Fig. 4.8. The figure shows that even in $MgSO_4$ 50mM solution (Mg^{+2} concentration in 96 percent sea water) the effect to nitrification rate was hardly seen, and also $(MgCO_3)_4$ $Mg(OH)_2$ was similar to MgSO4. Although there was about 10 mg/l of ammonia oxydation, there was no pH drop demonstrating strong buffer strength. Table 4.1 shows the population of bacteria after five minutes of each chemical addition.

Test 4) Effect of Anion under the Existence of Phosphate

Fig. 4.9 (a) and (b) show the ammonia oxidation rate of halogen concentration against control. The Figures show that the concentrations that decreased rate to 50 percent were NO^-_5(29mM) > F^-(48mM) > SO_4^{2-}(76mM) > I^-(80mM) > Cl^- (170mM) in that order. Cl^{-6-} had shown the least effect.

Fig.-4.6 Change in Population of Nitrifiers

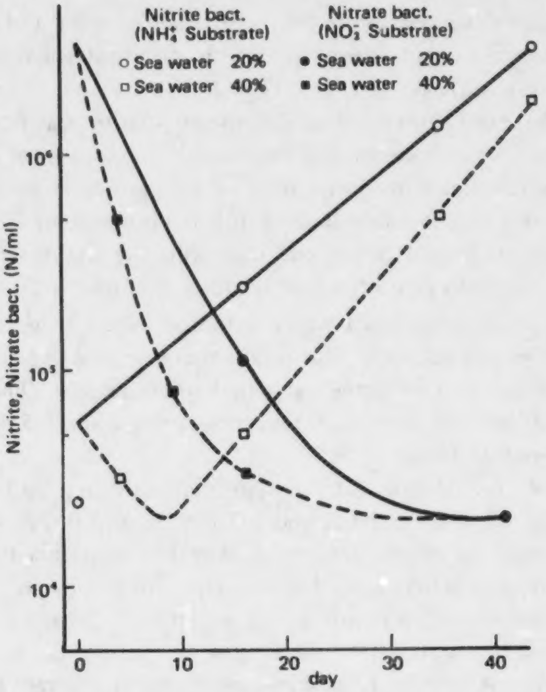
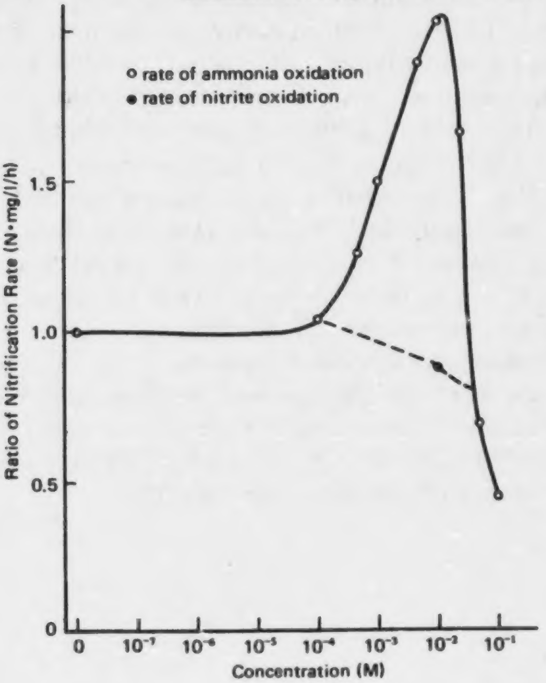


Fig.-4.7 Rate of Ammonia Oxidation, Nitrite Oxidation vs Phosphate Concentration



4.3 DISCUSSION

The Influence of Tris-buffer on Nitrification:

Despite the fact that the generation time of Nitrite bacteria by pure culture is known to be between 10 and 12 hours, it was 10.6 days in 20 percent sea water addition, 7.9 days in 40 percent solution or 23 and 17 times respectively longer than pure culture. Furthermore, from Figures 4.3(a) and 4.5, when comparing the lag phase of sea water addition of 0 percent and 20 percent, the former showed no difference, but the latter had shown an excessive difference of 14 days between 0 percent sea water (without Tris-buffer), perhaps, owing to the fact that Tris-buffer had caused inhibition to nitrite bacteria growth.

By addition of Tris-buffer, as is seen from Fig. 4.3(b), nitrite concentration increase significantly. The population of nitrate bacteria with NO^-_2 as substrate in Fig. 4.6, diminished with the passing of days. From these facts, it is concluded that Tris-buffer acted to inhibit activity of nitrate bacteria.

Addition of higher sea water had higher bacteria growth rate, and this phenomenon is reasoned as follows: Either substances contained in the sea water acted as catalyst to stimulate growth or metal enzyme, engulfed in bacteria, had been formulated to promote growth. According to Loveless Report (1968), by addition of $MgCl_2$, the nitrite bacteria hightened in growth. So, this fact in this study was considered to be due to Mg contained in sea water.

It is regarded that anions such as Cl^- and SO^-_4 affected to causing the acceleration of lag phase and nitrification rate in proportion to the increase in sea water addition. From the fact that the amount of nitrification per nitrite bacterium was smaller in 40 percent sea water solution, it is necessary to check whether the amount of nitrification per nitrite bacterium will increase by grafting and acclimatization.

With regards oxidation of ammonia to nitrite, phosphate proved effective as accelerators.

The forms when phosphate is pH 7.5 and pH 8.0 are as follows: pH 8 . . . $H_2PO^-_4$ 13.8 percent, and HPO^{2-}_4 86.2 percent. pH 7.5 . . . $H_2PO^-_4$ 33.9 percent and HPO^{2-}_4 66.5 percent (at 25 degrees centigrate). If $H_2PO^-_4$ is an accelerator to promote nitrification, it should increase 2.5 times between pH 7.5 and pH 8, and HPO^{2-}_4 should decrease more than 20 percent. With this in mind the subject rests were conducted by setting the pH at about 7.75, a high concentration phosphorous acid solution in which pH drop is scarce.

By the addition of more than 13mM of phosphate, the activity to convert to NO^-_2 suddenly decreased. But despite the fact an equal amount of nitrite bacteria inhabited in the solution until 10mM, the bacterial population decreased to about 1/3 in 25mM phosphorous acid solution. From these facts, it was considered that even though the apparent nitrification dropped suddenly, the per bacterium nitrification inhibition also slightly decreased.

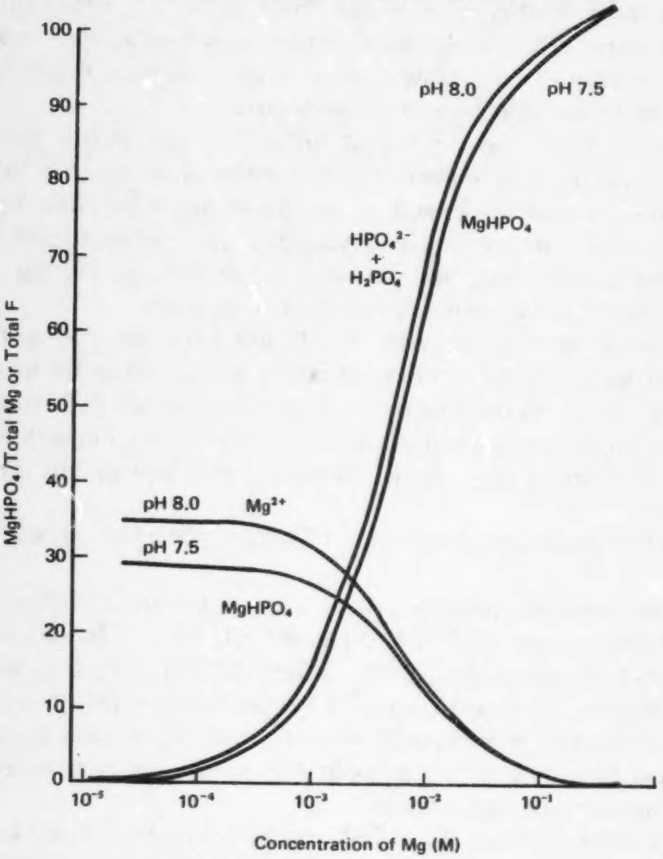
Characteristics of Cation Under the Existence of Phosphate:

It was proved that phosphate is an effective nitrification accelerator. But it bears a characteristic to form complex or sediments with cation. As an example, the characteristics of phosphate with Mg^{2-} are as shown in Fig. 4.10. The figure shows the existing form ratio of Mg^{2+} in 2mM of phosphate. The reaction of phosphate and Mg^{2+} is expressed in the following equation:

$$Mg^{2+} + HPO^{2-}{}_4 \rightleftharpoons NgHPO_4 \ (pK = 2.18)$$

From this equation, the complex formation is influenced by the concentration of phosphate. If phosphate concentration is maintained constant level, by increasing Mg^{+2}, the amount of complex increases. In this case, however, $MgHOP_4$ / Total Mg ratio will decrease.

Fig.-4.10 Mg^{2+} vs $MgHPO_4$ (Phosphate 2mM, at 25°C)



When studying the effects of additives such as cation, the addition of buffer solutions should be with care. Addition of the buffer solutions is held down to the minimum. For example, in case of phosphate, it should be controlled to about 0.1 μM. In this case, the nitrogen as substrate, too, must be similarly controlled.

However, it is impossible to conduct the aforementional studies by using activated sludge. When activated sludge is cultured, NH_4^+ will liberate from the sludge. And the release rate of NH_4^+ is affected by the composicion and concentration of the culture solution, the condition of the collected activated sludge (collected time & place), the preservation method of the collected sludge, and so on. In this study irrelevant to the concentration of phosphate, the increase in T-N due to the release of NH_4^+ within five hours was not observed. But 24 hours later, the soluble T-N, when compared to the initial T-N, increased about 5 (mg/l as N). Release of NH_4^+ was observed until about four hours in ohter case. But after that (between 4 and 6 hours) no release was observed. The initial release rate without any chemical addition was about 1.9 to 2.9 (mg.N/liter/h). Also, in proportion to the increase in anion addition the release of NH_4^+ showed an inclination to decrease. In this way, the cause of the large amount of NH_4^+ release in the initial stage is attributed to the preservation method of activated sludge. In other words, as the collected activated sludge (MLSS: 10,000 – 20,000 mg/l) was preserved under an aeration, the released NH_4^+ has nitrified and caused pH to drop. It should be noted if pH adjustment is conducted only once or twice every day, the nitrification bacteria will exist in the solution at around pH 5 most of the time. However, in this test as pH was maintained highly between 7.5 and 8.0, it was concluded that the release was not triggered by sudden pH changes.

From these results, in the nitrification tests using activated sludge, it is recommended that a reasonably large amount of nitrogen as substrate be needed.

4.4 SUMMARY

1) Under the existence of Tris-buffer, the population of nitrite bacteria contained in 20% sea water solution and 40% solution were counted and the growth rate in 40% solution was about 1.34 times faster than in 20% solution. This was, perhaps, caused by Mg^{2-} in the sea water. However, the amount of nitrification amount per nitrite bacterium was higher in the 20% sea water solution.

2) In short period incubation (under the existence of 2mM phosphate), no Mg^{2+} nitrification acceleration activity was observed.

3) In short period incubation, nitrification to nitrite was accelerated by phosphate addition and the optimum concentration was about 13mM.

4) Under the existence of 11mM phosphate, when expressing the 50% nitrification decrease in terms of concentration due to the influence of anion, it can be as in the following order: NO_3^- (29 mM) > F^- (48 mM) > SO_4^{2-} (76 mM) > I^- (80 mM) > Br^- (140 mM) > Cl (170 mM).

5. DETERMINATION OF NITRIFICATION BACTERIA AND HETEROTROPHS IN ACTIVATED SLUDGE

The biological composition of activated sludge is: bacteria, protozoa and metazoa or microorganisms. And bacteria exist in the form of autorotrophs such as nitrite bacteria, nitrate bacteria. Then there are the heterotrophs. Counting of protozoa and metazoa population is possible by the microscopic method. On the other hand, although there are no established measurement methods for bacteria, Sakurai (1967) and Chida et. al (1971) reported several types of culture media for the measurement of heterotrophs. With regards the measurement of autorotrophs, especially nitrite and nitrate bacteria, the plate culture method and the liquid culture medium are suggested. But concerning the development of the plate culture method incorporating silica gel, it takes up considerable time in the preparation of the culture medium and the colony is little too small for counting. Therefore, the method is considered inconvenient for practical use.

Incubation of nitrification bacteria is generally conducted by the MPN Method introducing the liquid culture medium. But this medium, too, has many problems yet to be cleared such as determination of the incubation period, etc.

In this study to bring to light the succession of nitrifiers and heterotrophs in the biological treatment system, considerations were made on the dispersing method, incubation period and culture medium of specimens that were considered fundamental to measure the actual population of existing microotganisms.

5.1 TESTING PROCEDURE

The specimen sludge was activated sludge produced by the indoor experiments boosting a comparatively high nitrification activity.

The TOMY SEIKO UP200P Ultrasonic Shredding Unit was introduced to disperse the specimen. The selected power for the apparatus was 25 watts with the maximum shredding time being 3 minutes. The specimen was a mixed solution of 5 ml in quantity. The cultured temperature was maintained at 20°C and the incubation periods for heterotrophs maximum 30 days and nitrifiers maximum 130 days.

Considerations of the culture medium were made only on heterotrophs. And concerning nitrification bacteria, there were no discussions conducted pertaining to the culture medium incorporating the IBP Method.

The culture media for heterotrophs were: Medium A . . . ordinary agar medium. Medium B . . . ordinary agar medium diluted 10 times. Medium C . . . Sakurai Culture Medium. Medium D . . . Trypton Glucose. Medium E . . . ASE Culture Medium. And Medium F . . . PWRI (Public Works Research Institute) Culture Medium. There were six types in all. As PWRI culture medium employs synthetic sewage produced by the indoor experiments, 15 grams of agar was added to the syncthetic sewage to have the concentration of influent. The population of heterotrophs was measured by counting all colonies formulated in the medium. With regards measurement of nitrification bacteria, the counting of nitrite and nitrate was done to measure nitrite bacteria; while the number of nitrate was counted and plus and minus judged to measure nitrate bacteria. The measurements of nitrite and nitrate were conducted by introducing the Technicon Auto Analyzer based on the Cd-Cu Reduction Method.

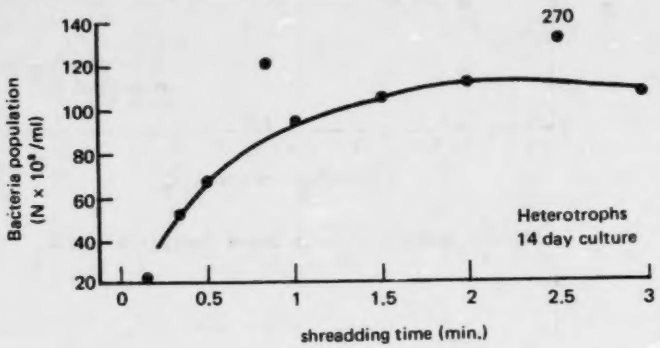
5.2 RESULT AND DISCUSSION

5.2.1 Dispersion of Specimen Sludge

Various types of microorganisms are adhered to the activated sludge. Especially, as nitrification bacteria are microorganisms with a sticking tendency (fixed type), special methods must be incorporated to disperse them to make way for easy measurement. And there are many methods developed.

Since the specimen contains live microorganisms, this gives rise to the problem of dispersion force. Therefore, it is important to confirm the optimum conditions of each specimen with regards dispersion method and force before implementing dispersion. Here, the ultrasonic shredding method was implemented and the relation between shredding time and the number of microorganisms was searched. The results are shown in Figs. 5.1 and 5.2.

Fig.-5.1 Living Microorganisms and Ultrasonic Shredding Time



Until around two minutes of shredding, the number of bacteria continued to increase and reached a fixed quantity, (two or three minutes.) Similar tendency was observed of nitrification bacteria. When this apparatus is used, the shredding time would be about 2.5 minutes. According to Chida (1971), with homogenizers, the appropriate time was 10 minutes. However, when coping with a number of specimens, the ultrasonic shredding unit is instrumental to shorten time. When conducting microscopic observation of specimens subjected to ultrasonic shredding, protozoa were shredded between 1.5 and 2 minutes leaving no trace of the original form. With regards shredding of Rotifera, only traces of shells were barely recognizable. From these observations, it is assumed that shredding can cause the mortality rate of the microorganisms to increase. Therefore, comparison with other dispersion methods is necessary. Here the shredding time was set at 2.5 minutes for activated sludge.

5.2.2 Incubation Period

The growth rate of bacteria is easily subjected to physico-chemical conditions such as temperature, pH, nutrient source and inhibitors. The growth rate is further influenced by other microorganisms in the mixed culture system. The culture temperature was set at 20°C, a temperature requiring longer incubation period. Concerning activated sludge and heterotrophs, the Gils (1964) at normally 25°C, the incubation period is 6 days. Tezuka (1968) reports at 30°C, the culture period is between 6 and 7 days. Chida et al. (1971) statee at 20°C, the incubation requires 10 days. And the IBP Method (1969) insists at 30°C, the period is between 7 and 10 days of incubation.

The relation between incubation period and the number of bacteria in this test was as shown in Fig.s. 5.3 and 5.4.

Fig.-5.2 Nitrification Bacteria and Ultrasonic Treatment Time

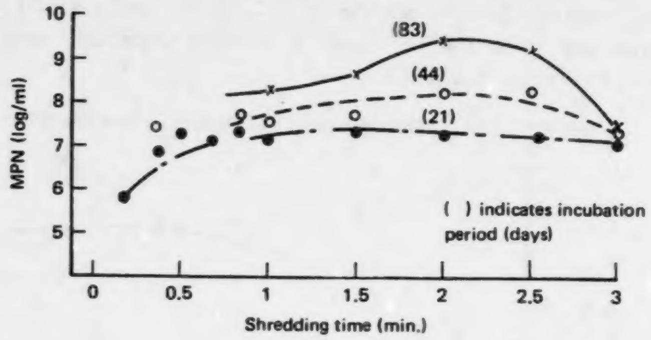


Fig.-5.3 Number of Bacteria and Incubation Period

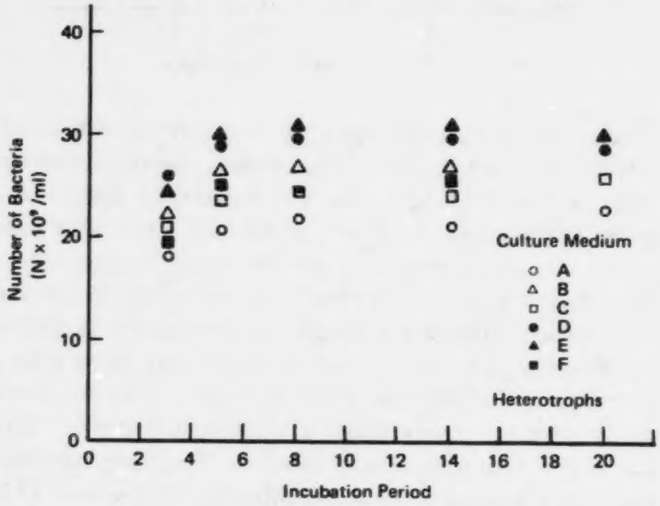
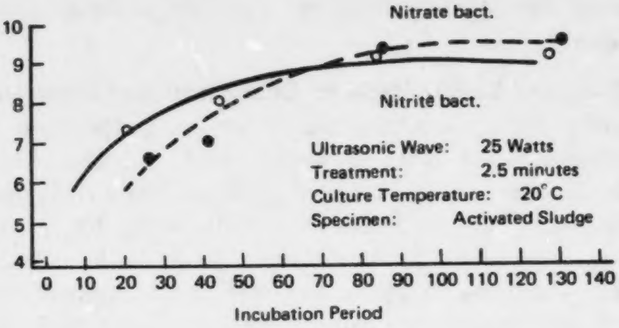


Fig.-5.4 Incubation Period and Number of Bacteria



The heterotrophs, after eight days, have shown fixed values. This tendency was also observed in other specimens. Accordingly, the culture of heterotrophs was conducted at 20°C and the incubation period being 10 days. With regards nitrification bacteria, Meikle John (1975) reports 25°C for two weeks. IBP states at between 20 and 22°C, the period is over 60 days. Matulewich et al. (1975) states that at 28°C, the incubation period for nitrate bacteria is between 20 and 55 days and nitrite bacteria between 108 and 113 days. When compared with heterotrophs, both the nitrate and nitrite bacteria require longer incubation periods. As shown in Fig. 5.4, at 20°C, the equilibrium condition was reached after more than 60 days. However, with the passing of days, after the 60th day, the nitrification bacteria showed a tendency to further increase. This inclination differed with the types of specimens, thus making the determination of incubation period difficult. Bacteria that turn positive after 100 days of incubation period, or those with extremely slow multiplying rate, were found impossible to propagate in the bilogical treatment system except under special conditions. In optimum condition, the generation time (tg) of nitrification bacteria is regarded to be between 10 and 12 hours. On the other hand, to estimate the incubation period by judging the number of days needed to reach the specified nitrification extent based on the growth rate of nitrifiers and the actual amount of nitrifiers when nitrites and nitrates produced by nitrification were detected, it is as follows:

From this study, it became obvious that the nitrification bacteria population more than about 10^3 and 10^4 N/ml, and thus the post-culture bacteria was estimated to be 10^4 N/ml. When incubation is conducted by the MPN Method, the nitrification bacteria amount would be 1 N/test tube of 5 ml at a certain dilution stage. From this, when the incubation period is seeked by setting the generation time at 12 hours, it will be about 8 days. In case of marine nitrification bacteria, Yoshida (1967) states the generation time would be 40 hours at 20°C while Kimata and his group (1961) insist that the incubation period will be mroe than 2 months. When the generation time is 40 hours, the incubation period will be 22 days. By considering this value and the logarithm derived from the environmental changes in the test tube by the MPN method, it may be appropriate to define those that turned positive at about 40 days of incubation as nitrification bacteria. The culture temperature should be between 25 and 27°C, or the optimum temperature of nitrification bacteria.

5.2.3 Studies of Culture Media for Determination of Heterotrophs

Various types of culture media are being developed for measurement of bacteria contained in activated sludge and in streams, lakes and swamps. Here, comparison of six different types of culture media was conducted. Fig. 5.5 shows the results of 14 days of incubation period. Except for specimen No.4, no drastic differences were observed. As shown in Photo. 1, the colonies in media E and F were small compared to others and made measurements difficult. Here, of the remaining four specimens, culture medium D enabled easiest measurement. Therefore, it was decided to introduce culture medium D (Trypton Glucose) for measurement of heterotrophs.

Fig.-5.5 Bacteria Number and Culture Media

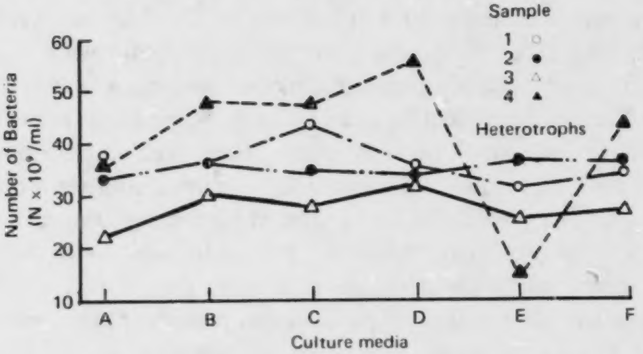
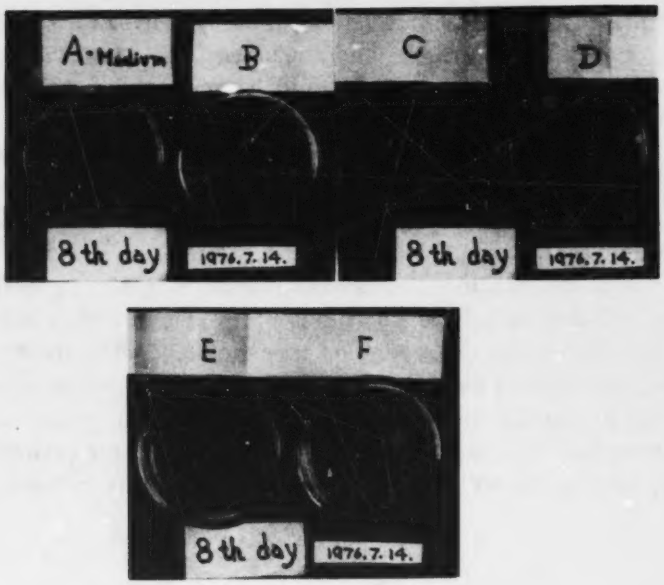


Photo-1 Comparison of Colony Size in Tested Culture Medias



5.3 SUMMARY

When conducting measurements of nitrification bacteria and heterotrophs contained in activated sludge, comparisons and studies of culture media were made only of dispersion method of sludge, incubation period and heterotrophs. The ultrasonic shredding unit was introduced to disperse sludge with an output of 25 w for 2.5 minutes. The incubation period was 10 days at 20°C for heterotrophs. However, with regards nitrification bacteria, there were difficulties in determining the incubation period. It was decided that the appropriate incubation period for nitrification bacteria was between 20 and 40 days based on tg, etc. Among introduced six different types of culture medium no drastic differences were observed. However, in view of the fact that the size of the colony was comparatively large and measurement easy, it is recommended adopting the Trypton Glucose culture medium.

II. FIELD SURVEY ON NITRIFICATION AT EXISTING SEWAGE TREATMENT PLANT

The studies were conducted to clarify the condition of nitrification at an existing activated sludge plant, in order to acquire a firm grasp of the relationships between nitrification and various contributing factors in specific reference to laboratory tests. Survey items include the nitrification rate, number of nitrifying bacteria, fauna, etc. in the aeration tank.

The hydraulic load at the plant surveyed is relatively low, and nitrate nitrogen is detected in its effluent almost throughout the year. The plant has a design capacity for treating 200,000 m^3/day of inflow, and its present inflow is approximately 150,000 m^3/day. This plant, Arakawa STP is located in Saitama Pref..

1. METHOD OF SURVEY

The survey was conducted using one bay of aeration tanks having a length of 85 m, with sampling conducted at four points equidistant from each other between the inlet and outlet of aeration tank. Sampling time was set at 11:00 hours when hydraulic load is considered to be relatively stabilized. Operational factors such as hydraulic load and MLSS were not controlled for this purpose, and survey was made once a month.

Water temperature, pH, DO, BOD, COD, N, nitrifying bacteria, fauna, are measured and analyzed.

The nitrifiaction rate was obtained in terms of the yield of nitrate and nitrite nitrogen per unit time during the period taking the highest rate. The lag time was taken as the time differential between the inlet of aeration tank and the formation of nitrate and nitrite nitrogen.

2. RESULTS OF SURVEY

Table 1 shows the summary of the 1977 survey results. As seen in the table, the water temperature ranged from 12.5 to 24.8°C, and the aeration time from 5 to 7 hours. SRT (sludge retention time) influencing the standing crop of nitrifying bacteria exceeded 14 days which, however, could have been longer because withdrawal of excess sludge was conducted not continuously but intermittently at a rate of once in three days or a week. pH generally maintained at the neutral level. BOD-SS loading recorded less than 0.1 g/ss·g/day in most cases, although a maximum of 0.16 g/ss·g/day was observed once. The aeration tank is relatively run at low loadings. However, the hourly variation of BOD-SS is unknown because 24-hours survey was not conducted. DO registered 0.3 ~ 5.6 mg/l. As this value was obtained by settling the sample for liquid-solid separation and by applying Winkler's azide modification method to the supernant, it is probable that DO in the aeration tank was slightly higher.

Judging the degree of nitrification on the basis of the above data, it can be said that the water temperature worked as the rate determining factor of the activity of nitrification. Hence, the data shown in Table 1 and 2 are employed as control data for clarifying other factors of nitrification in respect to water temperature and season.

Table-1 Sumurary of the Results (1977)

Date	77 5/10	6/13	7/13	8/11	9/13	10/11	11/7	12/13	78 1/9	2/7	3/27
W.T. (°C)	19.9	23.5	23.2	24.8	22.5	21.5	20.5	17.0	14.5	12.5	16.5
Aeration time (h)	5.87	6.59	5.48	5.93	4.56	5.25	6.04	5.74	6.22	6.84	5.86
pH	6.2	7.0	6.8	6.5	6.9	6.8	6.6	6.7	6.8	7.0	7.1
DO (mg/l)	3.3	0.5	0.3	1.2	5.6	4.4	0.3	0.3	0.7	1.0	3.1
MLSS (mg/l)	4,205	4,700	2,237	3,400	1,434	3,000	3,849	3,800	3,300	3,714	3,000
MLVSS (mg/l)	2,405	3,000	1,407	2,210	915	1,980	1,680	2,508	2,300	2,622	1,900
MLVSS/MLSS	0.57	0.64	0.63	0.65	0.64	0.66	0.70	066	0.70	0.71	0.63
BOD·SS Loading	0.06		0.16	0.03	0.06	0.03	0.04	0.05		0.07	0.09
COD·SS Loading	0.02	0.02	0.03	0.02	0.03	0.02	0.02	0.02	0.02	0.02	0.03
Nitrification rate (%)	60.1	27.0			14.5		50.2	30.3	49.5	25.5	12.3
Nitrite bacteria (N/ml)	4.9×10^5	11×10^5	21×10^5	10×10^5	23×10^5	13×10^5	23×10^5	13×10^5	13×10^5	0.2×10^5	21×10^5
Nitrite bacteria (N/ml)	2.3×10^6	1.7×10^6	13×10^6	0.8×10^6	0.5×10^6	4.9×10^6	23×10^6	3.3×10^6	11×10^6	0.3×10^6	4.9×10^6
SRT (d)	69	15	18	32	24	100	60	24	20	26	14

Table-2 Summary of the Results (1976)

Date	76 5/27	6/16	7/12	7/21	8/9	9/13	9/27	10/7	11/24	11/30	12/20	77 1/17	2/21	3/16
W.T. (°C)	19.3	19.3	21.8	22.2	24.8	21.2	21.0	21.9	17.8	16.7	15.7	13.6	12.5	16.8
pH	6.6	6.7	6.8	7.0	6.9	6.9	6.7	6.8	6.95	7.01	6.99	6.95	6.97	7.02
DO (mg/l)	6.7	4.9	6.8	7.1	9.5									
MLSS (mg/l)	3,500	2,000	2,000	3,000	2,300	1,500	2,500	2,500	4,490	1,960	2,680	2,520	2,510	2,330
MLVSS (mg/l)	2,230	1,180	1,300	2,040	1,580	880	1,680	1,540	3,160	1,350	1,840	1,900	1,800	1,880
MLVSS/MLSS	0.64	0.59	0.65	0.68	0.69	0.59	0.67	0.62	0.70	0.69	0.69	0.75	0.72	0.81
BOD·SS Loading	0.047	0.094	0.034	0.023	0.027	0.020	0.053	0.064	0.032	0.091	0.14	0.17	0.12	0.22
Nitrification rate (%)	64.3	71.6	83.8	72.1	92.4	64.9	90.2	82.7	96.6	87.1	55.9	30.5	31.8	15.7
Aeration time (h)	5.10	5.10	5.10	4.08	4.08	5.10	4.08	5.10	4.76	5.10	5.10	4.37	5.31	5.31
SRT (d)	57	35	71	53	31	62	94	31	71	49	62	134	122	68

As for the population of nitrifying bacteria in activated sludge, Kobori et al. indicated in their report (1977) that it registered approximately 10^6 N/ml and generally maintained itself at this level when SRT was more than 10 days (20°C), and that the number of heterotrophic bacteria was $10^8 \sim 10^9$ N/ml.

In the present survey, SRT ranged from 14 to 100 days and the bacterial number determind, through the year, $10^6 \sim 10^7$ N/ml for nitrate bacteria and about 10^6 N/ml for nitrite bacteria (Fig. 1). As shown in Fig. 2, about the same bacterial numbers were detected from the results of extended aeration process. These facts indicate that where SRT longer than the generation time of nitrifying bacteria is maintained, the bacterial number in activated sludge is held at about 10^6 N/ml irrespective of SRT. While the number of heterotrophic bacteria in activated sludge is known to range from $10^8 \sim 10^9$ N/ml, that of nitrifiers is much smaller, ranging from 10^2 to 10^3 N/ml.

Fig. 3 shows the relationship between nitrification rate and water temperature. Although it is required to obtain the nitrification rate per unit bacterial number, the figure indicates the rate per MLSS as the 1976 survey excluded the measurement of

Fig.-1 Seasonal Variation of Nitrifying Bacteria in Aeration Tank

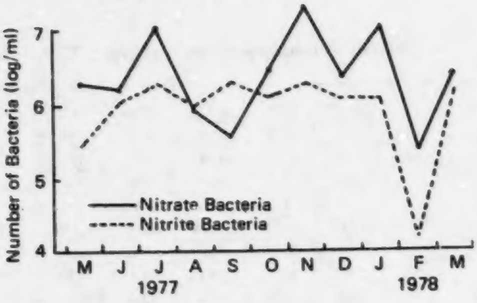


Fig.-2 Seasonal Variation of Bacteria in Aeration Tank (Extended Aeration Plant)

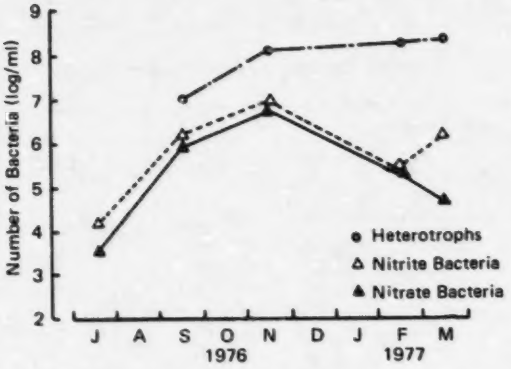
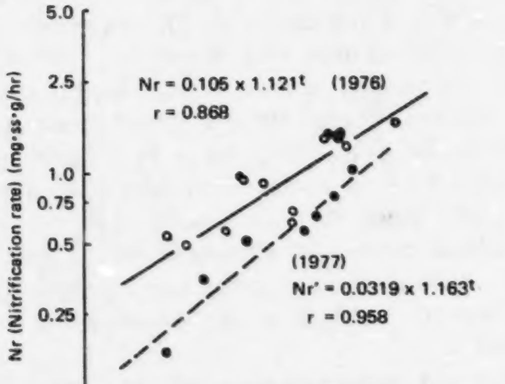


Fig.-3 Effect of Temperature on Nitrification Rate



bacterial number. As seen in this figure, the nitrification rate rose slightly when the operating factors in the aeration tank were controlled. The rate declined under normal operating condition, which is perhaps assignable to the influences of other factors than water temperature. The conceivable influences of such other factors are loading variation and DO level, and they seemed to become greater with the decline of water temperature. Temperature coefficient obtained on the basis of the observed data was a value of 1.121 under controlled operating condition (1977 survey) and 1.163 under non-controlled condition (1976 survey). These values show fair conformity with those in relevant reports.

Observation of nitrification which takes place with the downflow in the aeration tank disclosed that it involved certain time lag. Fig. 4 showing the relationship between water temperature and lag time indicates that latter tends to become longer with the decline of the former. The exact reason of this lag time increase is unknown, but it appears probable that as the activity of nitrifying bacteria becomes weaker due to the drop of water temperature, the influences of other environmental factors become stronger. Reduction of lag time which is often evidenced in laboratory tests is essential for efficient utilization of aeration tank, and this is one of the most important problems to be solved for satisfactory nitrification at an actual plant.

Fig.-4 Effect of Temperature on Lag Time

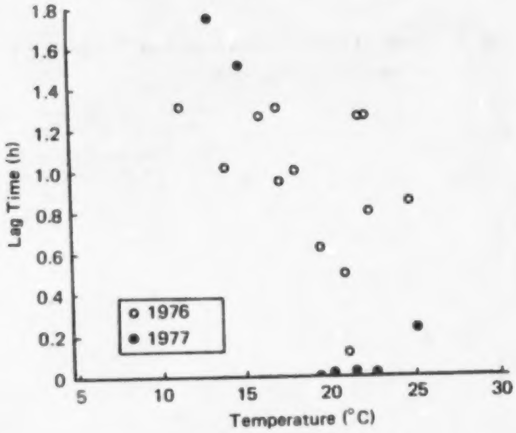


Table 3 shows the fauna in activated sludge. The fauna was predominated by activated sludge type maintaining activated sludge in a favourable condition throughout the year. Euglypha and Rotifera whose propagation is accelerated by the operational condition favourable for nitrification were observed through the year. At present, studies are being made on the possibility of using Euglypha as an indicator of nitrification condition in the aeration tank.

Table-3 Protozoa Observed Frequently during the Study

N/ml

Date	76 5/10	6/13	7/13	8/11	9/13	10/11	11/7	12/13	77 1/9	2/7	3/27
Litonotus	40	40	591	100	40		620	400	820	229	1,480
Arcella	1,060		617	1,160	460		340	1,560	1,460	7.233	1,460
Euglypha	480	340	711	1,400	120		640	380	40	130	100
Aspidsca	180	20	720	240	280		880	940	1,360	3,862	760
Vorticella	1,900	60	231	2,840	840		3,480	1,080	320	4,123	140
Espitylis	640	1,420	103	620	620		2,120	1,960	1,060	9,492	20
Opercularia	220		180	60	520		220	340	200	164	60
Acinata	60		51	80	20		180	80	60	65	60
Rotifera	280	140	695	2,900	340		40	340	340	229	240
W.T. (°C)	19.9	23.5	23.2	24.8	22.5	21.5	20.5	17.0	14.5	12.5	16.5

Fig. 5 shows the effect of nitrification rate and water temperature on the population of Euglypha which is one of the Protozoa detected when SRT is long. It was found that the number of Euglypha increased/decreased in proportion to the nitrification rate and water temperature, its coefficient of correlation with water temperature being 0.64. From this fact, it was discovered that the number of Euglypha is directly indicative of the degree of nitrification in the aeration tank. This relationship also applies to a plant employing extended aeration process as it was noted that Euglypha was dominant when the NH_4-N concentration in the effluent was less than 1 mg/l.

Fig.-5 Effect of Nitrification Rate and Temperature on Number of Euglypha

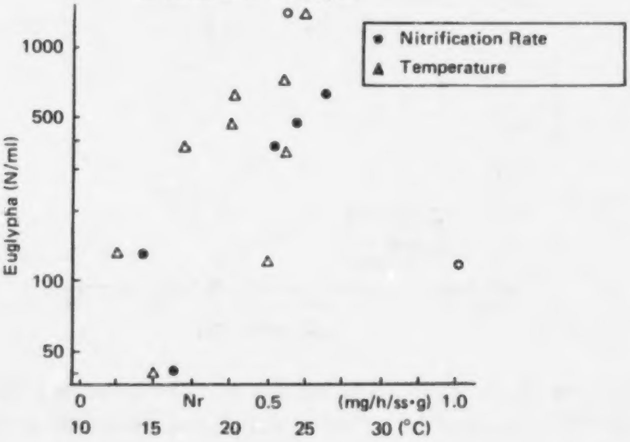
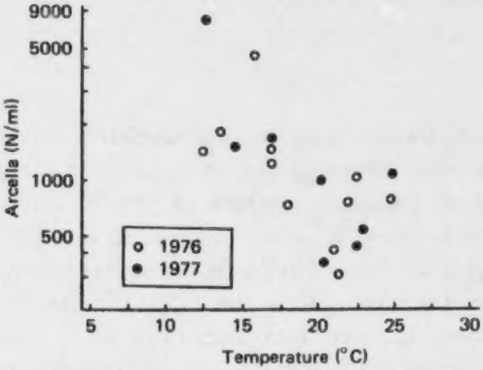


Fig. 6 shows the relationship between water temperature and population of Arcella. As seen in the figure, the number of this amoeba was inversely proportional to water temperature at a correlation coefficient of 0.72. Arcella is observed, just as Euglypha, when SRT is long, but it increases with the decline of water temperature.

It is planned that sutdies on index fauna will be continued as the operation and maintenance of aeration tanks can be performed with a mininum of time and labour by the use of such fauna as indicators.

Fig.-6 Relation between Temperature and Number of Arcella



3. SUMMARY

The findings of the survey introduced above can be summarized as follows.

1) Number of nitrifiers in activated sludge is on the order of 10^6 N/ml.

2) While there a diversity of factors influencing the nitrification at existing sewage treatment plants, the nitrification rate can be maintained at a high level by controlling environmental factors affecting it and by operating the plant as constant load as possible.

3) Nitrification reaction in the aeration tank involves a lag time, which is an important matter to be considered for designing and operating nitrification plant.

4) For the purpose of satisfactory maintenance of aeration tank, its operating condition can be determined by the bacterial population (biomass) and aeration time on the basis the relationship between nitrification rate and water temperature. Also, as mentioned, the operating condition needs to be determined with consideration of the lag time involved in the nitrification.

5) There is probability that Euglypha will be used as an indicator of the degree of substantial nitrification.

III. NITRIFICATION ACCELERATION BY ALUM ADDITION TO THE PRIMARY SEDIMENTATION TANK

INTRODUCTION

To induce nitrification, it is necessary to accelerate growth of nitrifying bacteria in the treatment process. When compared with other bacteria related with water treatment, nitrifiers are comparatively slow in growth. Therefore, to accelerate the growth of nitrifying bacteria, it is indispensible to lengthen the retention time of micro fauna existing in the treatment process to help growth of nitrifiers.

In the activated sludge process, the SRT (Sludge Retention Time) of the activated sludge process is a significant factor to control. The SRT is expressed as a ratio between the amount of solids existing in the treatment process and the amount of solids discharged outside of the treatment process as effluent SS or waste activated sludge. Therefore, to extend the SRT means either to increase the amount of solids in the treatment process or to decrease the amount of solids discharged.

To maintain solids in the treatment process, there are means to elevate the return sludge rate, to prepare a sludge storage tank and others. To decrease discharged solids, considerations can be made to either lower the influent organic load or the influent SS load that will in turn decrease the amount of solid production.

This report is a summary of experiments that were aimed at accelerating nitrification by extending the SRT as a result of suppressing solid production by lowering the organic load in the aeration tank after adding alum as a coagulation agent to the primary sedimentation tank.

1. OUTLINE OF THE FACILITIES

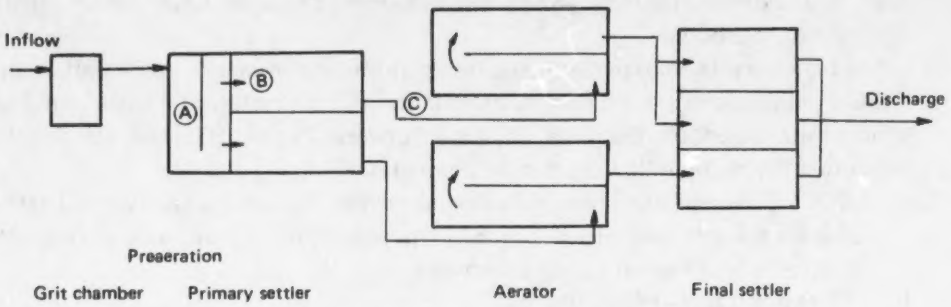
The studies were conducted at the Nishiyama Sewage Treatment Plant in Nagoya city. With treatment capacity of 20, 000 ~ 30,000 m^3/day, it is a plant handling influent composed of only domestic sewage. Fig. 1 shows the facilities and the flow-diagram.

From the primary sedimentation tank to the final sedimentation tank, it can be broken up into two independant bays – Bay No. 1 (the lower part of Fig. 1) and Bay No. 2.

In experimental phases No. 1 and No. 2 alum was added to the preaeration tank while the primary effluent was supplied to the aeration tank in the Bay No. 1, and the primary effluent was added with NaOH and its pH values adjusted before being send into the aeration tank in the Bay No. 2. In phase No. 3, the addition of alum to the primary sedimentation tank was stopped in the Bay No. 1, and continued only in the Bay No. 2.

Furthermore, in the Bay No. 1, except during phases No. 1 and No. 2, the conventional activated sludge phases No. 1 and No. 2, the conventional activated sludge process was employed in its operation. On the other hand, in the Bay No. 2 prior to the subject experiments, tests were conducted on adding alum to the aeration tank.

Fig.-1 Flow Diagram of Nishiyama STP



Point of alum dose	Ⓐ ··············· Phase No. 1, No. 2 Ⓑ ··············· Phase No. 3
Point of NaOH dose	Ⓒ

2. INFLUENCE ON TREATMENT DUE TO ADDITION OF ALUM TO PRIMARY SEDIMENTATION TANK

First, the experimental conditions for both bays during the subject experiments are listed in Table 1. The MLSS values that show a great difference between the two bays in Table 1 was considered to be the difference in the capacity of the final sedimentation tanks.

Table-1 Sumary of Operational Conditions

Phase	No.1 1977. 9.13 ~ 10.15		No.2 10.15 ~ 11.9		No.3 11.9 ~ 1978. 1.26	
Bay	No. 1	No. 2	No. 1	No. 2	No. 1	No. 2
Flow (m^3/d)	14,570	14,570	13,920	13,920	12,180	12,180
Aeration time (hrs)	2.55	2.06	2.65	2.13	2.96	2.32
Return sludge rate (%)	20.6	49.4	21.6	51.7	24.6	59.1
Air flow (times)	1.6	3.3	1.7	3.4	2.0	3.9
MLSS MLVSS (mg/l)	703 562	2125 1382	687 502	1999 1361	831 643	2607 1832
SVI	97	63	98	69	113	90
BOD·SS load (kg/kg/d)	0.450	0.149	0.688	0.236	0.666	0.153
SRT (d)	6.3	37.6	2.9	13.4	1.9	13.6
Alum does as Al (mg/l)	6	6	3	3	0	6
NaOH addition	No	Yes	No	Yes	No	Yes

With regards the influence of the alum addition to the primary sedimentation tank, it was divided into two groups: the influence to primary treatment and that to the secondary process.

Concerning the influence to the treatment at the primary sedimentation tank, Table 2 summarized the primary sedimentation effluent values and the removal rate which were based on the data obtained between March 1975 and March 1978. From this Table, the followings can be pointed out:

a. BOD, COD and TOC removal rates improved. Especially, the removal rate of soluble matters markedly improved. With regards the removal rate of suspended matters, there was no significant change.
b. SS removal rate hardly changed.
c. Phosphorous removal rate changed markedly from 17.4% to 51.9%.
d. Although slightly, K-N and NH_4-N removal rates increased.
e. pH values dropped around 0.3 and 0.5.
f. Total alkalinity of influent without alum addition increased slightly. But with addition, it dropped a little.

Besides the above mentioned points, the increase in primary sludge production and the improvement of primary influent transparency can be quoted.

Table-2 Changes in Primary Effluent Quality with Alum dose

	No addition		Alum addition (6 mg Al/l)	
	Effluent Quality	Removal Rate	Effluent Quality	Removal Rate
BOD (T)	73.4	42.1 (%)	51.5	58.4
BOD (S)	43.7	14.1	29.4	47.2
COD (T)	43.2	35.5	28.6	51.9
COD (S)	22.0	5.2	——	——
TOC (T)	52.2	30.0	41.4	40.2
TOC (S)	26.6	12.2	22.2	30.7
K-N	23.1	9.7	19.2	25.1
NH_4-N	13.5	-5.9	11.6	4.6
T-P	3.08	17.4	1.84	51.9
SS	46.7	67.5	47.1	66.6
T-Alkalinity	89.6		52.9	
pH	6.6 ~ 7.3		6.3 ~ 6.8	

(mg/l)

Concerning the influence to secondary treatment, data obtained between mid September and mid November were collected and compared. The results of data comparasions are shown in Table 3. However, the 1975 and 1976 data are of operations by the conventional system and 1977 of operations by adding alum to the primary sedimentation tank.

From Table 3, the followings can be pointed out:

a. The effluent values and the removal rate of BOD and COD did not show much changes.
At BOD (T), the improvements on the removal rate and the effluent values were perhaps due to the fact that oxygen consumption by nitrification in a BOD bottle was small.

b. With regards TOC, the removal rate deteriorated.
The reasons are yet unknown.

c. Nitrification virtually did not occur, and K-N, NH_4-N removal rate dropped.

d. With regards phosphorous, both residal concentration and removal rate increased.

e. Due to drop in waste activated sludge production, the SRT became longer.

Table-3 Secondary Effluent Quality when Alum is Added in Primary Settler

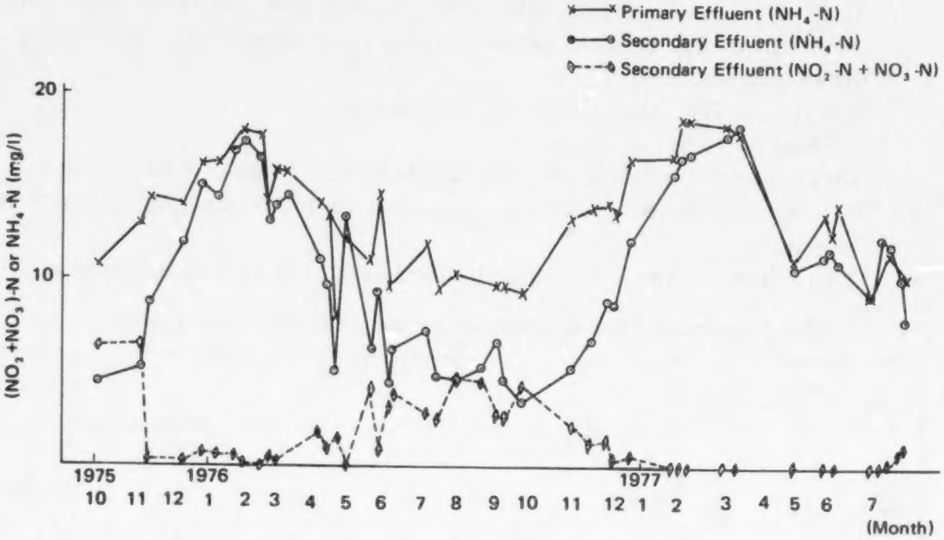
Year	1975		1976		1977	
	Effluent quality	Removal rate	Effluent quality	Removal rate	Effluent quality	Removal rate
BOD (T)	19.5	68.3(%)	27.2	55.9(%)	11.8	77.4 (%)
BOD (S)	3.7	89.8	7.6	79.5	5.7	81.5
COD (T)	11.1	71.7	13.2	73.5	10.8	62.4
COD (S)	7.5	55.3	9.3	57.7	——	——
TOC (T)	11.8	65.5	10.1	71.9	23.0	54.0
TOC (S)	8.2	59.7	——	——	19.0	33.6
K-N	8.6	61.3	7.9	60.8	14.8	24.4
NH_4-N	5.4	58.2	5.4	56.5	10.7	9.5
NO_2-N + NO_3-N	5.5	——	3.4	——	0.7	——
T-P	1.62	37.7	1.60	37.3	0.65	61.8
SS	9.8	77.6	13.2	65.9	6.8	82.2
MLSS	1227		730		646	
SRT (d)	——		3.4		4.6	
Mode of Operation	Conventional		Conventional		Alum addition in primary settler	

3. ACCELERATION OF NITRIFICATION

3.1 NITRIFICATION BY CONVENTIONAL PROCESS

Fig. 2 shows the actual results of the Bay No. 1 demonstrating to what extent can nitrification be expected in the conventional operating process.

Fig.-2 Seasonal Variation of Nitrogen at Conventional Process



The NH_4-N concentrations in both the primary sedimentation tank effluent and the secondary treat ment effluent were higher in winter and lower in summer, thus showing seasonal changes. The difference between the two, in other words the NH_4-N removal rate (nitrification rate), was approximately 0% in winter and between 50% and 60% in summer.

3.2 THE EFFECTS OF pH ADJUSTMENT WHEN ADDING ALUM TO PRIMARY SEDIMENTATION TANK

The SRT during summer when nitrification developed was between 2.5 and 3.5 days by conventional operation. By adding alum to the primary sedimentation tank, sufficienct length was obtained with regards SRT. But, despite this fact, nitrification was insufficient, perhaps, due to the drop of pH value in the aeration tank. It is known that the activity of nitrifying bacteria reaches its peak when pH are between 7 and 8. But, the pH value in the primary effluent dropped to 6.3 ~ 6.8 when alum was added, and it is assumed that the value has further dropped owing to nitrification activity. Therefore, in the Bay No. 2 tests were conducted by adjusting the pH values with NaOH for the purpose to highten the pH and to give sufficient alkalinity for nitrification.

NaOH was added at the primary sedimentation tank, and pH adjustment was made according to the pH meter installed in the lower stream. The target pH value was between 7.8 and 8.2. Fig. 3 shows the influence of pH adjustment to the pH value in the aeration tank. This shows how the pH values changed at the end of the aeration tank with regards non pH adjustment (Bay No. 1) and pH adjustment (Bay No. 2). The Fig. shows that when pH adjustment was not conducted, the pH value dropped markedly from night to the following morning. The minimum pH value recorded was about 5.0. In contrast, when pH adjustment was conducted, there was no pH value drop at night, and the value was maintained throughout the day at between 6.0 and 7.0.

Fig.-3 Effect of pH Adjustment

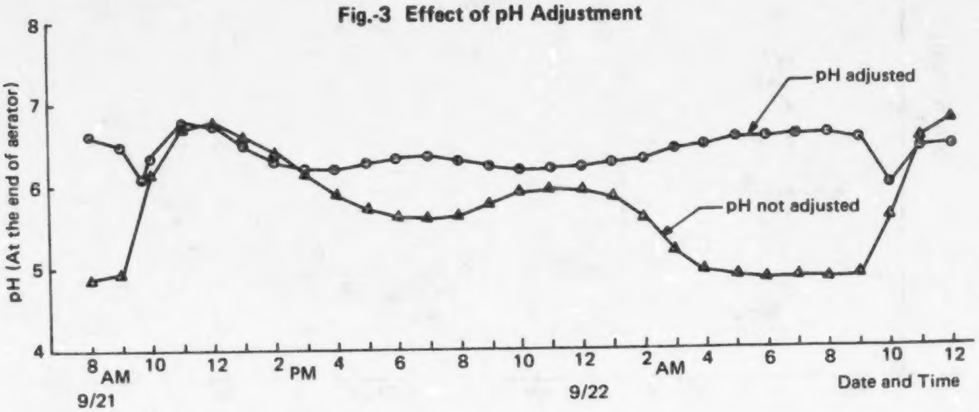


Fig. 4 shows the daily changes in NH_4-N concentration in the effluent of both the Bay No. 1 and the Bay No. 2. The Fig. summarizes the measurements conducted by the automated continuous colorimetric analyzer. The average of around-the-clock measurements from 9:00 am to 8:00 am the following morning is plotted as the value of a single day.

In the Bay No. 2, the daily average of 2 ~ 5 mg/l of NH_4-N in the effluent continued until mid-December, and from the latter part of December it gradually increased.

In the Bay No. 1, it can be pointed out that nitrification virtually did not develop when compared with that in the Bay No. 2 and in the conventional Bay No. 1.

From these results, even when SRT is sufficient, if the pH is too low, it can be anticipated that the nitrification will not be achieved. Therefore, an experiment was conducted. The results of this test are as shown in Fig. 5. As it was not possible to monitor the effluent in the Bay No. 2 at that time, it was substituted by expressing the changes of NH_4-N in the effluent of the filters. The influence of suspending pH adjustment have shown immediate results on the following day as the daily average of 1.6 mg/l was boosted to 6.6 mg/l. Also, the recovery after resuming pH adjust was fast and returned to normal in a signal day. This testified to the fact that the effects of pH adjustment were great and showed immediate results.

Fig.-4 Change in Ammonia Nitrogen

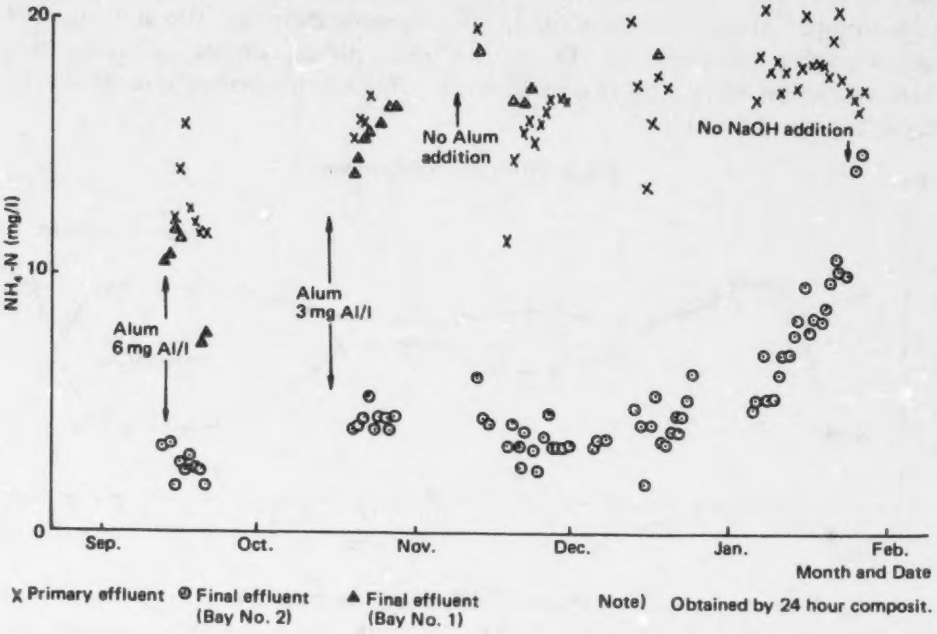
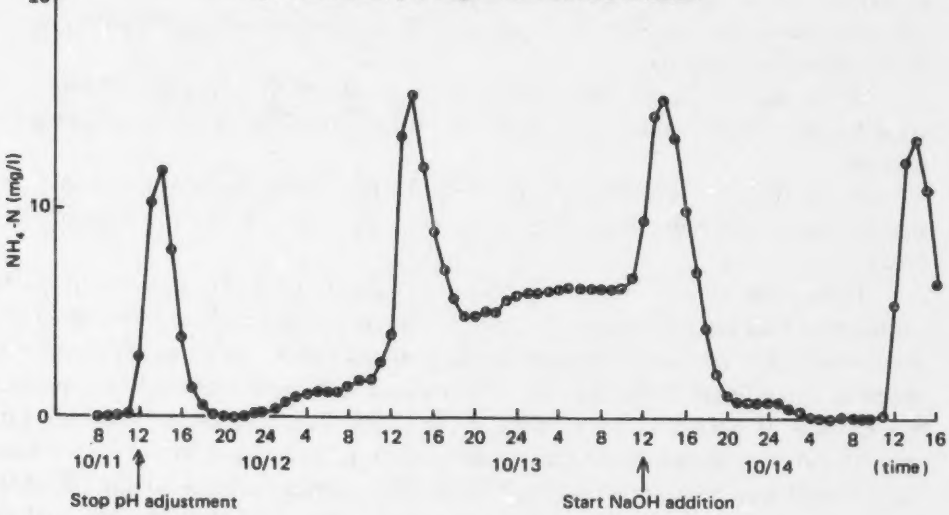


Fig.-5 Effect of pH Adjustment on Nitrification



3.3 NITRIFICATION AT HIGH LOADING

The NH_4-N concentration in the effluent varied greatly in a signal day. This is illustrated in Fig. 6. As shown in the Fig., except for the 9 hours between 11 a.m. and 8 p.m., the NH_4-N was totally nitrified and marked 'zero' in the Bay No. 1. The peak time during 11 a.m. and 8 p.m. came a little later than the peak time of primary effluent owing to the inflowing NH_4-N being discharged without totally nitrified.

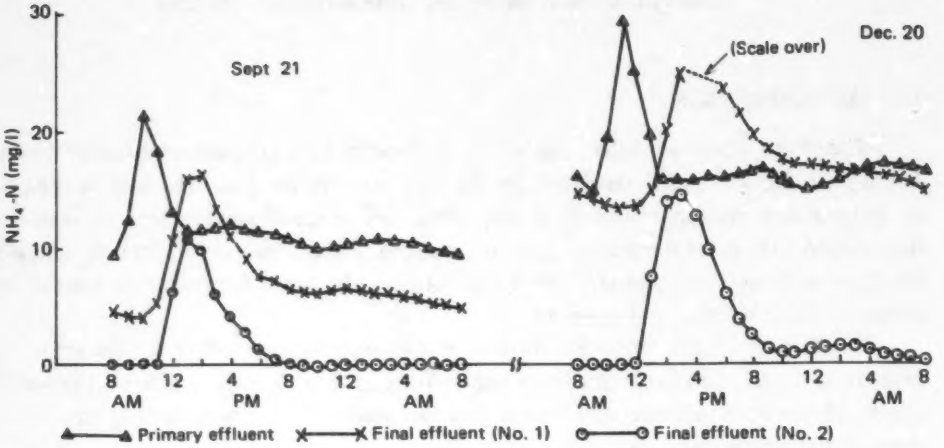


Fig.-6 Hourly Variation of Ammonia Nitrogen

During other hours than the peak time, nitrification was sufficient, thus showing that nitrification was insufficient during hours when either the influent volume or the influent concentration increased. It is regarded necessary to contrive means to heighten MLSS at least during high loading time.

This trend remains the same even in winter.

3.4 NITRIFICATION IN WINTER

When studying the Bay No. 2 data as shown in Fig. 4. nitrifiaction continued even in December and the nitrification rate began to drop from the latter part of December. Similar to the seasonsal changes observed in Fig. 2, this is considered to derive from the drop in the water temperature.

In the Bay No. 1 as shown in Fig. 2, it has lost its nitrification ability in January. In contrast, in the Bay No. 2, it maintained a nitrification rate of between 40 and 50 percent even in the latter part of January showing the strong influence of the SRT length.

It is considered difficult to prevent drop in nitrification ability due to water temperature drop despite sufficient SRT and pH adjustment.

4. SUMMARY

The results of the study are summarized as follows:

a. The arganic loading to the secondary process can be decreased and the SRT can be extended when alum is added to the primary sedimentation tank.

b. To extend SRT is necessary to induce nitrification. When SRT is sufficiently long, nitrification activity can be kept high even in winter.

c. pH adjustment was necessary to obtain completely nitrified effluent.

d. Control measure to increase MLSS at high loading is nescessary to maintain substantial nitrification.

IV. STUDIES ON NITRIFICATION PROCESS AT SMALL-SCALE SEWAGE TREATMENT PLANT

1. INTRODUCTION

The Nasu plant used for this study is located in a coppice of Kuroiso town, Tochigi Prefecture and is designed for treating wastewater from the service area of an expressway running through a hilly area. Its operation is subject to seasonal fluctuation. Users of the service area increase particularly in periods including many holidays such as early January, May and August. In these periods, raw sewage increases in both volume and strength.

Wastewater comes from the restaurant and water closet in the service area. At present, only the final effluent from the plant is used as flushing water on the water closet, although a mixture of effluent and tap water (1 : 1) was used at the initial stage of the plant operation.

Sewage treatment at this plant is conducted by the combined application of activated sludge process and filtration. Final filtered effluent is discharge into a nearby river through an agricultural waterway. As the effluent is used for irrigation, its target quality is set at less than 5 mg/l for BOD, SS and NH_4-N. Considering the functions of the plant, this target can be attained for BOD and SS. Hence, removal of NH_4-N or nitrification is the main objective of sewage treatment at the plant.

This report introduces the existing state of the plant facilities, with specific reference to the nitrification process. As for nitrification denitrification process, studies are being made to obtain further detailed data.

2. OUTLINE OF THE PLANT

The plant is operated to treat raw sewage from the restaurant and water closet by the extended aeration process in order to attain the effluent standards shown below.

A portion of the effluent is reused as flushing water for the water closet, and the remainder is discharged into the agricultural waterway.

1) Effluent Standards

BOD	Less than 5 mg/l
SS	– Ditto –
NH_4-N	– Ditto –
E. coli group	Less than 3,000 N/ml

2) Design Flow and Water Quality of Raw Sewage

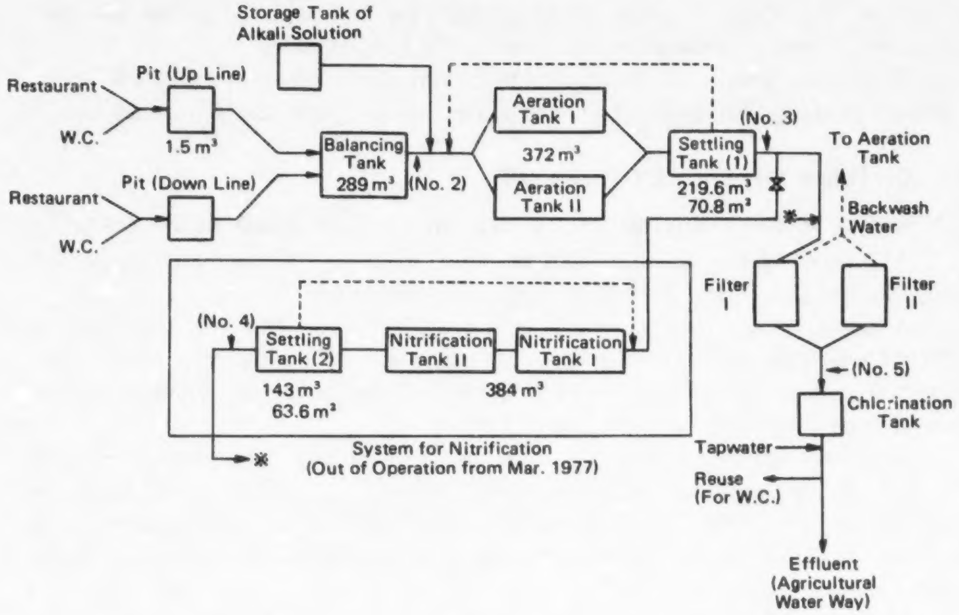
Average daily flow	860 m^3/day
Maximum hourly flow	89.6 m^3/h
BOD	200 mg/l
SS	250 mg/l

3) Treatment Process (Fig. 1)

The plant is designed for the combination of extended aeration and filtration. As shown in Fig. 1 illustrating the initially planned flow chart, the plant is composed of balancing tank (flow equalization), aeration tanks and settling tank for removing organics, nitrification tanks and settling tank for oxidizing nitrogen compounds, filters for removing solids, and chlorination tank.

Retention time of both aeration tanks and nitrification tanks is about 10 hours at the design flow. Actual flow is about a half of design flow. Dual media filters of sand and anthracite are used.

Fig.-1 Outline of the Plant



1) Operation by Initially Proposed Flow Chart
At the early stage of plant operation, treatment was carried out according to the initially proposed flow chart. However, as the detention time of aeration tank turned out to be more than 20 hours, nitrification occurred in the aeration tank, resulting in the reduction of pH down to about 5. In the nitrification tank, on the other hand, activated sludge was found only to a negligible extent and its segmentation caused turbidity increase. Hence, it was found meaningless to operate the nitrification tanks at that mode.

2) Proposed Operation for Improvement of Nitrification Efficiency
In order to raise the nitrification efficiency as well as the operational efficiency of the whole plant, the plant configuration was improved from March 1977 by suspending the operation of the nitrification system (central boxed section in Fig. 1) and by starting pH control in the aeration tank. The nitrification tanks thus put out of operation were used for aerobic sludge digestion.

3) Further Improvement for Nitrification-Denitrification
In March 1978, the plant configuration was further improved by introducing nitrification-denitrification process aimed at removing total nitrogen. This im-

provement, intended basically for introduction of a single stage nitrification-denitrification process, consists of the following main alterations.

The aeration tanks were converted to unaerobic tanks, a denitrification zone was formed by dosing methanol in the middle portion of the nitrification tanks, and end portion of the tank was used for re-aeration.

Studies are in progress on this introduced process.

Method of Survey

The survey, covering a period from January 1976 to March 1977, was conducted in two ways, 24-hour composite sampling and analysis carried out once week and 24-hour grab sampling conducted once in two months.

The survey points are shown in Fig. 1, and survey items included pH, transparency, alkalinity, SS, BOD, COD, nitrogen in various forms, fauna, bacteria, etc.

3. OUTLINE OF SURVEY RESULTS

3.1 RESULTS OF TREATMENT BY INITIALLY PROPOSED FLOW CHART

As a result of surveys conducted on March 1 and 2, 1976 according to the initially planned flow chart, the following findings were obtained.

Although organic matters such as BOD and COD were sufficiently removed, the effluent registered a transparency of 20 ~ 40 cm and contained micro flocs which were not removed by filtration. These particles were of the size small enough to pass through the filter paper pore size of 12 μ.

Residual concentration of NH_4-N ranged from 3.3 to 4.4 mg/l and averaged 3.7 mg/l. In the weekly sampling analysis, it recorded a value of 2 ~ 13 mg/l. At that stage of operation, tap water mixed with effluent as dilution water to meet standard, and the diluted effluent was resued as flushing water of water closet in service area.

pH was reduced from 6.8 ~ 7.6 (average : 7.4) to 3.9 ~ 4.0, and alkalinity registered zero. In the weekly 24-hour survey conducted, however, effluent pH showed considerable fluctuation, ranging from 3.8 to 6.6. Because of the low pH of effluent, alkali solution is added at the outflow side of the nitrification tank.

Nitrification rate recorded 13 ~ 78%, which is indicative of the possibility of conducting sufficient nitrification despite the water temperature of 8 ~ 10°C. Judging from the decline of pH, it appeared that the nitrification was supressed due to the shortage of alkali required for oxidation of ammonia.

Scum was formed in large quantities in aeration tanks and nitrification tanks. Microscopic examination disclosed that the sucm was air bubbles entrapped in oily film. The scum disappeared after its pH was adjusted to 8 ~ 8.5. Activated sludge presented a fauna predominated to a substantial degree by Rotifera and Euglypha, which is characteristic of the extended aeration process.

As for the functional performance of flow balancing tank, aeration tanks and filters, the following findings were obtained.

1) The balancing tank equiped with diffusers, is a fairly useful facility in that it can mitigate loading fluctuations by virture of its large capacity and is also capable of turning raw sewage aerobic (raw sewage in balancing tank contains

protozoa).

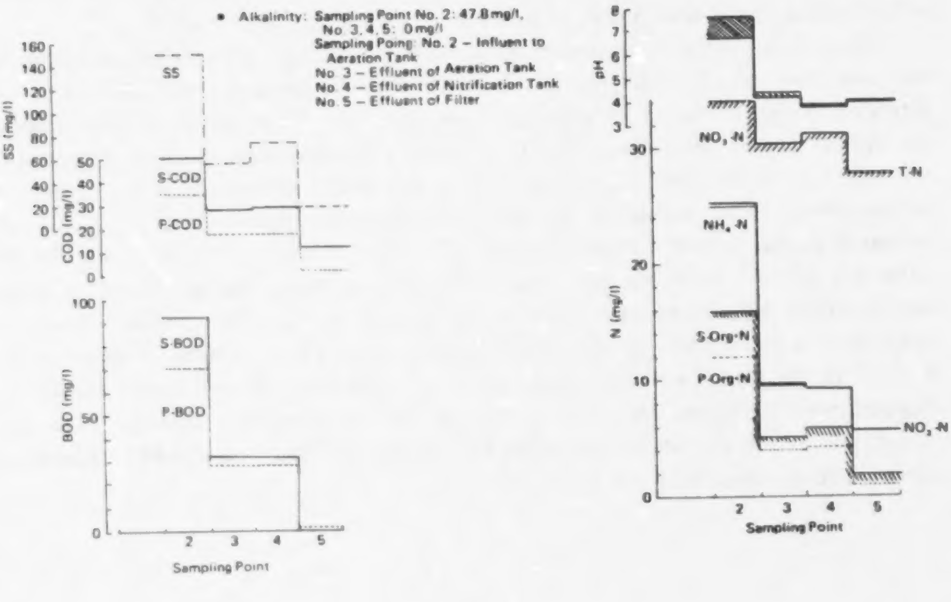
2) The aeration tanks suffice for removal of organic matters and oxidation of nitrogen as the aeration time is 24 hours at the present flow. However, the low pH in the tanks needs to be adjusted.

3) The nitrification tanks are not necessary at present because the aeration tanks suffice for both removal of organic matters and oxidation of nitrogen (Fig. 2).

4) The filters are indispensable for the extended aeration process like this plant. Specifically, the extended aeration process brings activated sludge close to the state of endogenous respiration, often causing flocs of activated sludge to be segmented or nitrified. This results in the denitrification in the settling tank, whereby nitrogen gas in produced to cause activated sludge to float up and overflow the weirs. Even in such a case, the filters are useful in improving the effluent transparency.

From the findings described above, it is concluded that 1) pH in the aeration tanks should be adjusted to about 8 in order to hold NH_4-N concentration in the final effluent at less than 5 mg/l at all times, and 2) considering the present flow, the nitrification tanks can be put out of operation without any hindrance to the plant operation.

Fig.-2 Average Water Quality of Each Process (1) Average Water Quality of Each Process (2)



- Alkalinity: Sampling Point No. 2: 47.8 mg/l,
  No. 3, 4, 5: 0 mg/l

Sampling Poing: No. 2 – Influent to Aeration Tank
No. 3 – Effluent of Aeration Tank
No. 4 – Effluent of Nitrification Tank
No. 5 – Effluent of Filter

3.2 PROPOSED IMPROVEMENT MEASURES BASED ON INITIAL OPERATIONAL CONDITION

1) pH Control

For the purpose of controlling pH, NaOH was selected for addition as 25% solution due to its nature. The injection dosage, about 60 kg/day, was determined for oxidizing TKN totally to nitrate so that the target pH of 8 ~ 8.5 would be attained by the plant operation.

2) Modification of Treatment Process

The nitrification tank was put out of operation for use as aerobic digestion tanks, and the treatment process was consequently modified as follows.

Raw sewage → Balancing tank → Aeration tank →
Settling tank → Filters → Reuse or discharge

After effecting the above two measures, the survey was continued with the dual purpose of removing organic matters and attaining 100% nitrification efficiency.

3.3 RESULTS OF SURVEY OF PROPOSED IMPROVEMENT MEASURES

3.3.1 Effects of pH Control

Summary of the survey results in shown in Figs. 3.1 ~ 3.3. By effecting pH control, removal rate of organic matters recorded 93 ~ 98% for BOD and 71 ~ 88% for COD. Removal of SS at secondary settler was also increased, but these filtered effluent showed the similar water quality as it did before pH control.

SS concentration in secondary effluent declined after pH control, although it was unknown whether this was to be ascribed to the increased bio-coagulation of activated sludge or to the shortening of aeration time. Particularly notable among the effects of pH control was the fact that the transparency of final effluent increased to more than about 1 m as a result of augmented SS removal in the filters. On examining SS in secondary effluent, it appeared that fine segmented particles of activated sludge flowed out in a sort of colloidal state when pH ranged from 4 ~ 5. After pH control, however, such fine particles disappeared and secondary effluent contained particles large enough to be entrapped in the filter, which is perhaps assignable to the shortening of aeration time by about a half. When pH ranged from 4 to 5 in the aeration tank, micro flocs in secondary effluent could hardly be trapped even if filtered by GFC. It may as well be added here that the minimum size of particles that can be entrapped by dual media filter of sand and anthracite is reported to be larger than 7 ~ 10μ.

Fig. 3-1 Summary of the Results

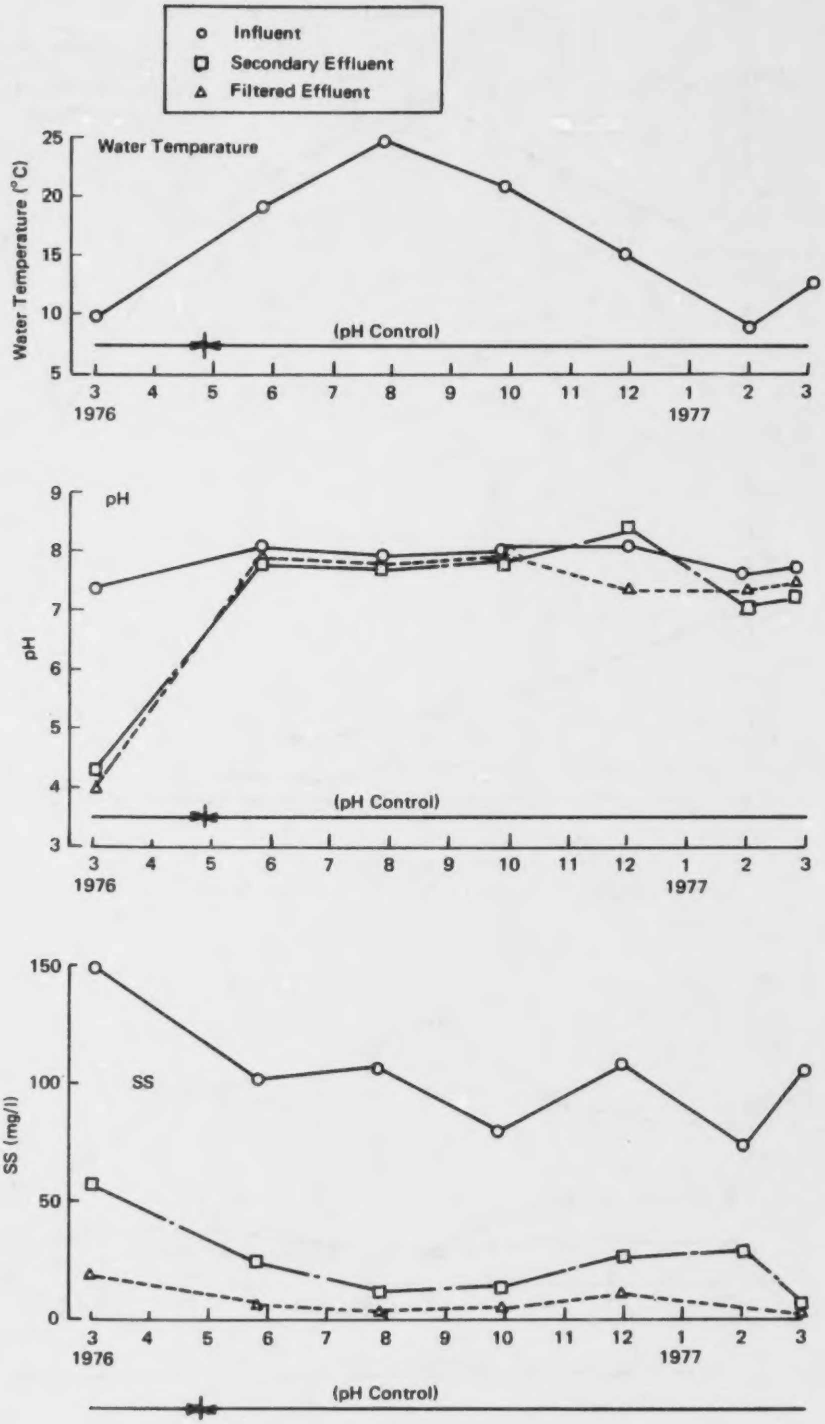


Fig.-3.1

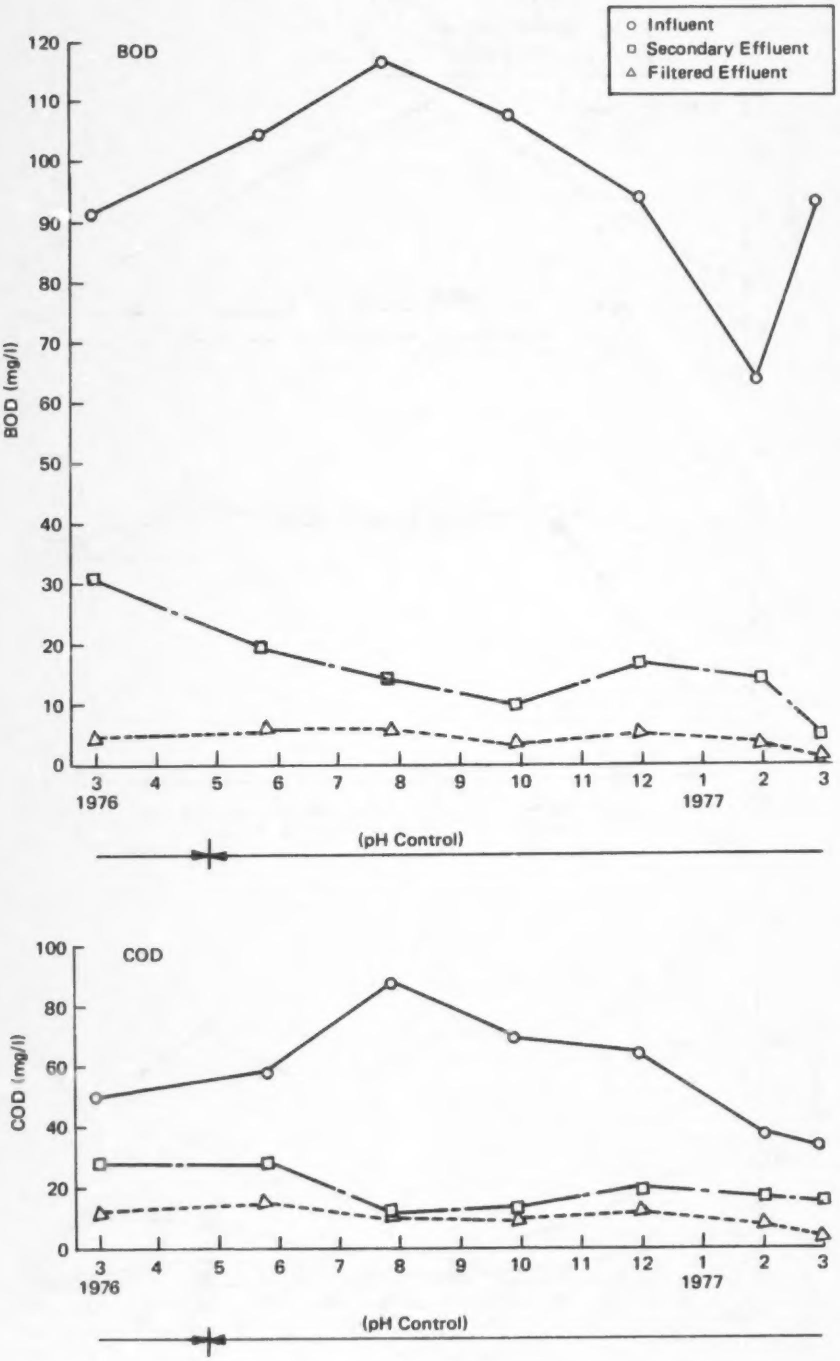

○ Influent
□ Secondary Effluent
△ Filtered Effluent
BOD
BOD (mg/l)
120
110
100
90
80
70
60
50
40
30
20
10
0
3
1976
4
5
6
7
8
9
10
11
12
1
1977
2
3
(pH Control)
COD
COD (mg/l)
100
80
60
40
20
0
3
1976
4
5
6
7
8
9
10
11
12
1
1977
2
3
(pH Control)

Fig.-3.3

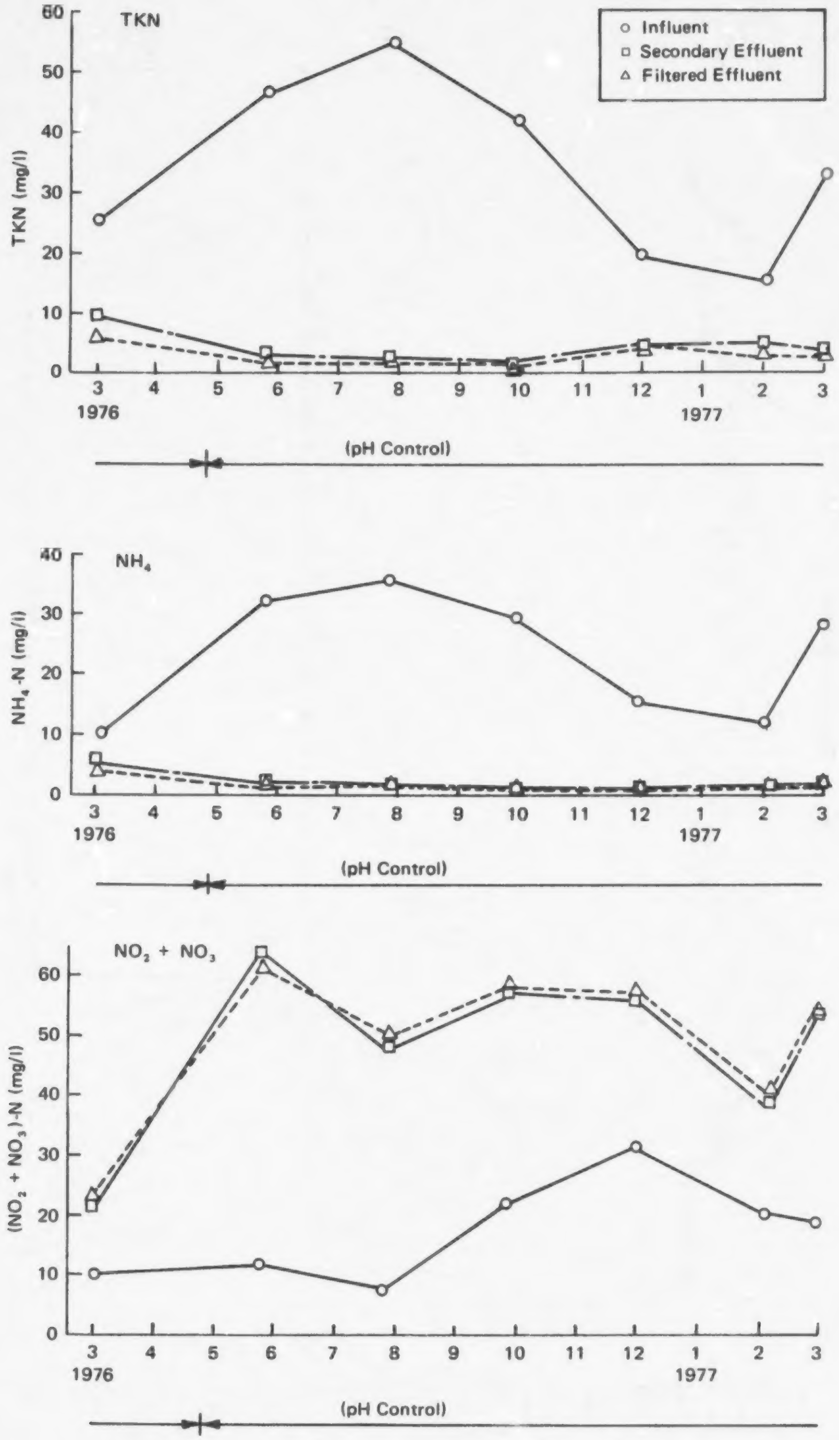
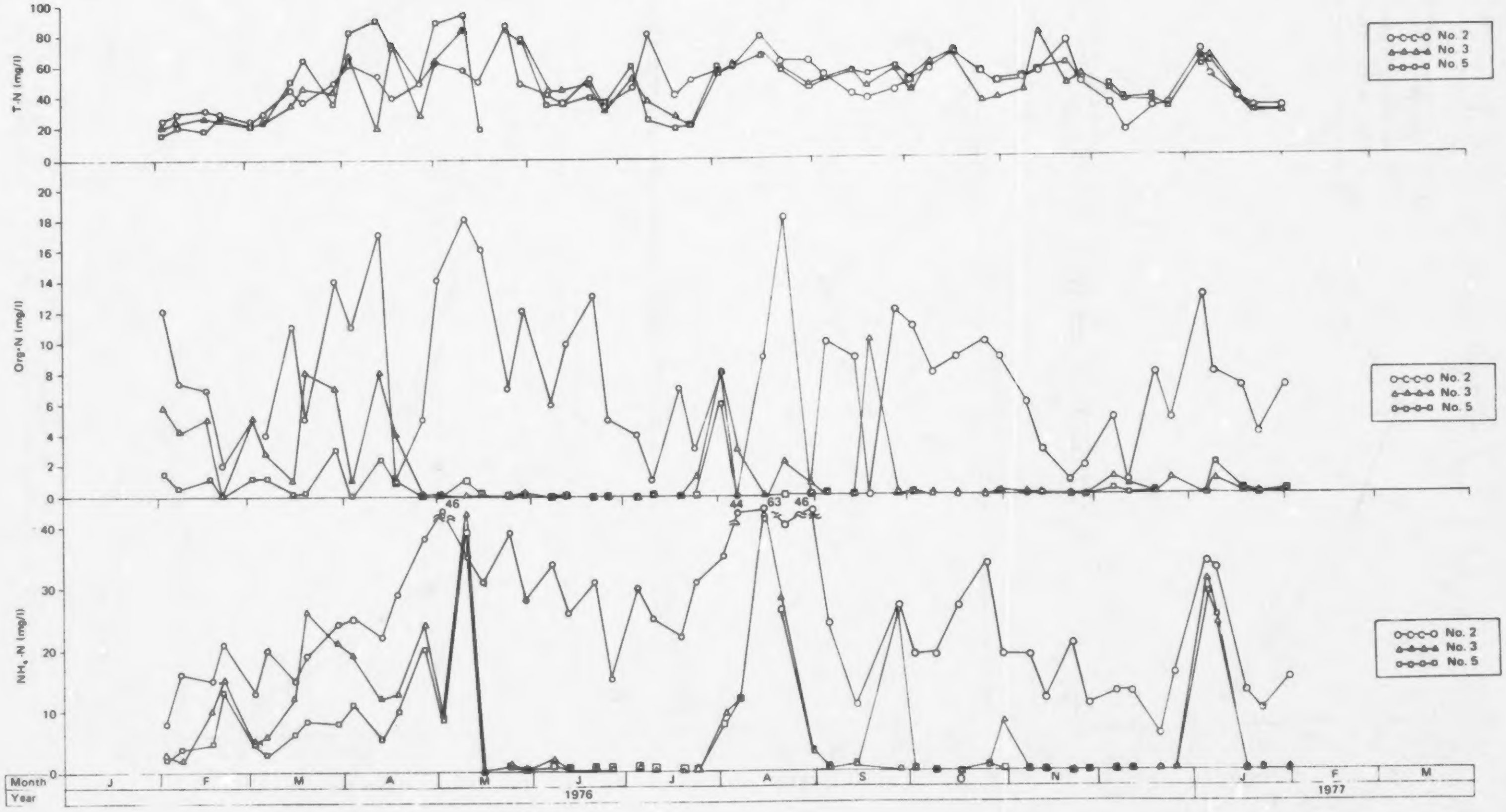

TKN
○ Influent
□ Secondary Effluent
△ Filtered Effluent
TKN (mg/l)
0
10
20
30
40
50
60
3
4
5
6
7
8
9
10
11
12
1
2
3
1976
1977
(pH Control)
NH₄
NH₄-N (mg/l)
0
10
20
30
40
(pH Control)
NO₂ + NO₃
(NO₂ + NO₃)-N (mg/l)
0
10
20
30
40
50
60
(pH Control)

Fig. 4 Results of Survey



As for nitrogen, TKN and NH_4-N in secondary effluent dropped to less than 5 mg/l and to about 1 mg/l, respectively, indicating that pH control exhibited a high nitrification efficiency. However, the weekly 24-hour composite survey (Fig. 4) disclosed that the nitrification rate declined to 30 ~ 40% in August and January when the aforementioned service area was most frequently utilized, with NH_4-N in final effluent recording 41 mg/l in August and 25 mg/l in January. A diversity of reasons can be conceived of as contributing to this phenomenon, such as the shortening of aeration time due to increased inflow of wastewater, decline of activated sludge density, sharp changes in pH, inflow of toxic or harmful substances, etc. However, since none of them can be identified as the decisive reason, further studies must be conducted to cast light on this problem.

The plant operating factors are shown in Table 1. The aeration time is as long as 20 ~ 40 hours. Even if inflow increases to designed 800 m^3/day, therefore, an aeration time of about 10 hours can be secured, which is long enough for removal of organic matters and nitrification under normal condition. Settling time, ranging from 12 to 24 hours, is also long enough to make the supernatant water considerably transparent, provided that activated sludge is condensed satisfactorily in the settling tank. MLSS is maintained at 1,200 ~ 3,400 mg/l, averaging 3,000 mg/l as seen in the table. BOD•SS loading is about 0.02 kg/kg/d operational factors discussed above, it may be said that the plant has considerably large surplus capacity.

Table-1 Operating Factors

	Date	Inflow (m³/d)	Aeration Time (hr)	Settling Time (hr)	Overflow Rate (m³/m²/d)	Discharge Loading (m³/m²/d)	MLSS mg/l	Inflow BOD Loading (kg/d)	Inflow SS Loading kg/d	BOD·SS Loading (kg/ss·kg/d)
1	1976 3.1~2	380.2 (15.9 m³/h)	23.5	13.9	5.4	12.7	1,670	14.3	56.6	0.02
2	5.24~25	223.8 (9.3)	40.0	23.5	3.2	7.5	3,140	14.0	23.5	0.01
3	7.26~27	292.1 (12.2)	30.6	18.0	4.1	9.8	2,950	18.9	31.3	0.02
4	9.27~28	297.1 (12.4)	30.1	17.7	4.2	10.0	2,620	13.3	23.8	0.01
5	11.29~28	401.8 (16.7)	22.2	13.1	5.7	13.5	2,900	26.4	43.4	0.02
6	1977 2.14~15	440.6 (18.4)	20.3	12.0	6.2	14.8	2,940	21.2	31.5	0.02
Average		339.3	27.8	16.4	4.8	11.4	2,703 *(1,910)	18.0	35.0	0.02

*pH Control

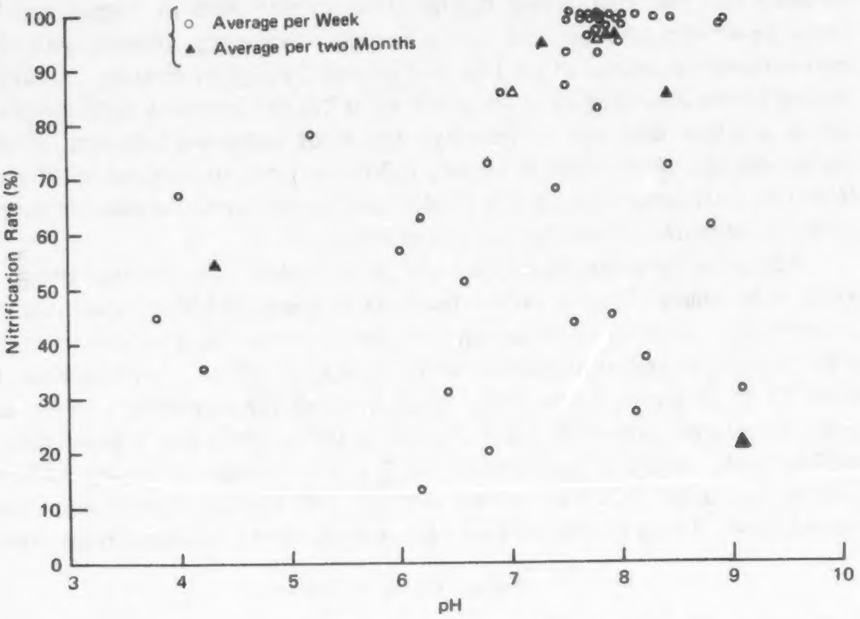
3.3.2 Relationship between pH and Nitrification Rate (Fig. 5)

Optimal pH for nitrobacteria is known to be 8 ~ 8.5. As for the relationship between the nitrification rate and pH, opinions differ from researcher to researcher, though to a minor extent.

In one of the papers dealing with this subject, it is reported that a nitrification rate of more than 90% can be obtained within a pH range of 7 ~ 9, which was confirmed by the experiment conducted at the Public Works Research Institute.

During the present survey, a nitrification rate of more than 85% was observed within a pH range of 7 ~ 9 except in May, August and January. When pH was lower than 7, the highest nitrification rate observed was about 80%. For satisfactory operation of the plant, therefore, it is advisable to maintain pH at about 7.5 ~ 8.

Fig.-5 Effect of pH on Nitrification Rate



3.3.3 Relationship between Water Temperature and Nitrification Rate (Fig. 6)

The effect of water temperature on the nitrification efficiency is not simply but influenced by the retention time of sludge in the treatment plant (SRT), so that simple comparison of the two is liable to produce a misleading conclusion.

In the case of this plant where SRT is very long, however, activated sludge maintains its activity of nitrification at a high level and pH remains constant after its control. As this makes it possible to regard that all environmental conditions excluding temperature are free from variation, the nitrification rate was compared with water temperature.

Before effecting pH control when the water temperature ranged from 8 to 13°C and pH was at a low level, the nitrification rate was subject to a large fluctuation, ranging from 13 to 86%.

After pH control, the nitrification rate recorded more than 90% except in May, August and January while the water temperature ranged from 8 to 25°C, indicating that the water temperature had practically nothing to do with the nitrification rate. Hence, the fluctuation of nitrification rate before pH control can be assigned not to the influences of water temperature but to the shortage of alkalinity. The survey also disclosed that the residual concentration of NH_4-N held itself at less than 1 mg/l for the greater part of the year (Fig. 7).

Fig.-6 Effect of Temperature on Nitrification Rate
('76. II ~ '77. III)

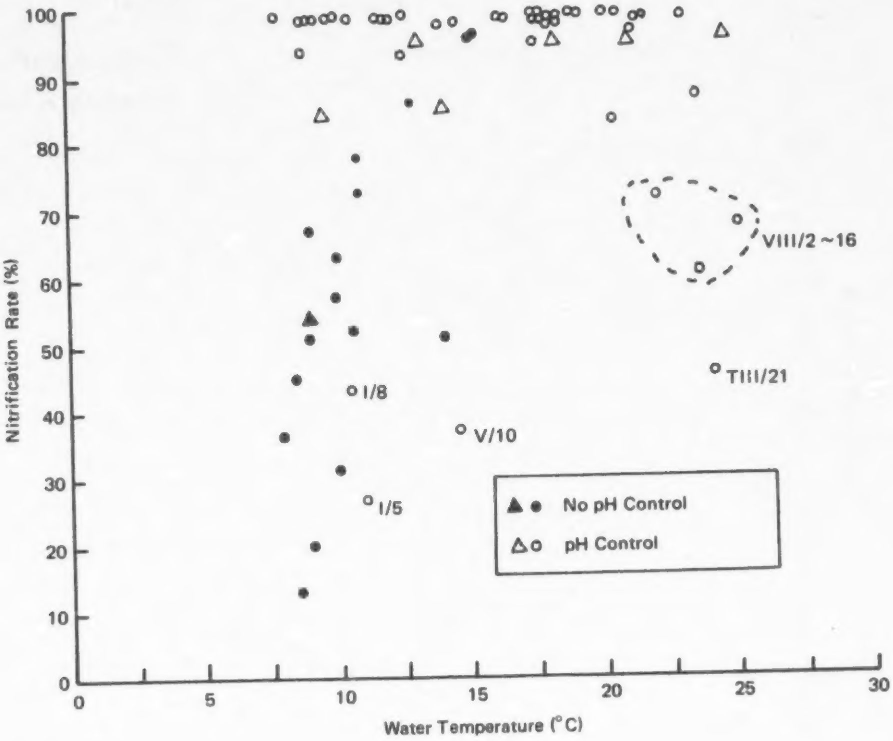
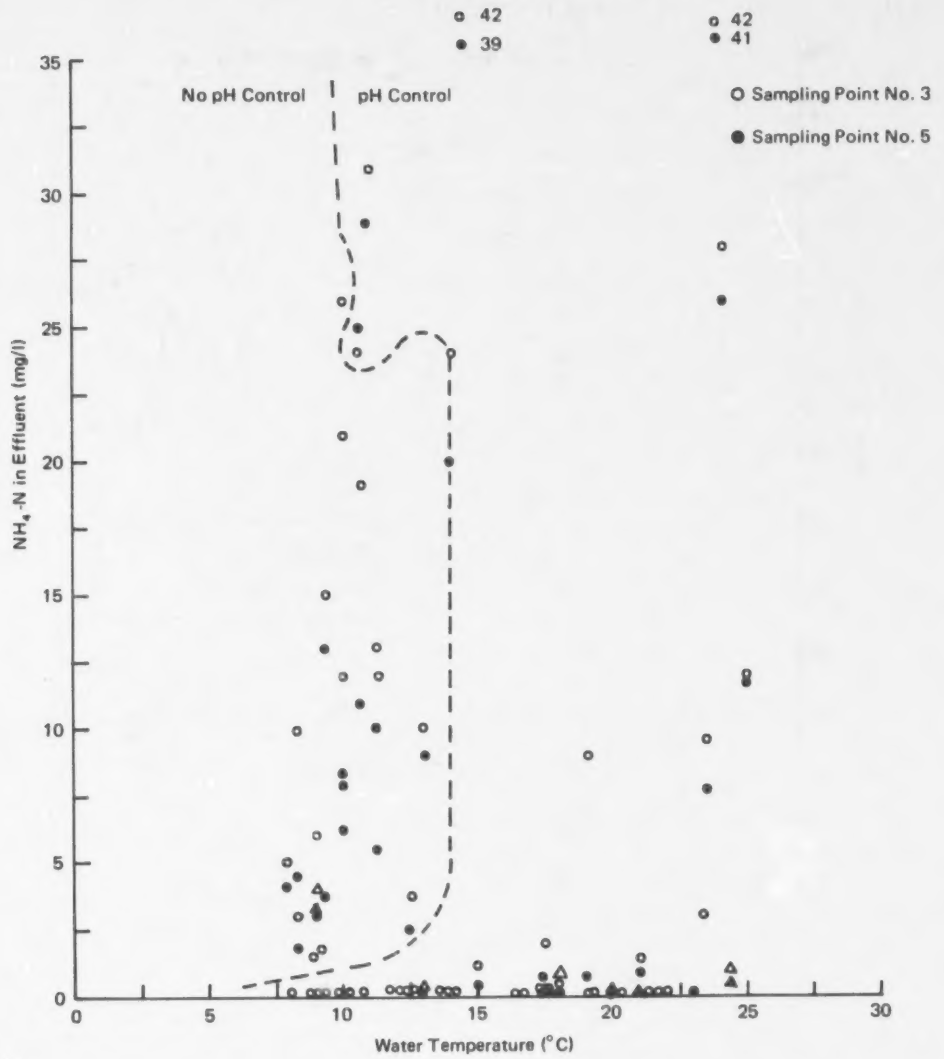


Fig.-7 Effect of Water Temperature on NH_4-N in Effluent



3.3.4 Relationship between pH and Total Alkalinity, and Effect of Alkalinity on Nitrification Rate (Figs. 8 and 9)

The relationship between pH and alkalinity as disclosed by the survey indicates that, unlike the values obtained from the neutralization titration curve, pH changed only to a limited degree even when alkalinity presented heavy fluctuations. Specifically, pH maintained a level of about 7.5 ~ 9 when alkalinity ranged from 160 to 400, which indicates that overdose of NaOH will not exert any adverse influence on biological treatment process. As pH can be helf at 7.5 ~ 8 if alkalinity of about 180 ~ 200 is present, it will be possible to save the chemical cost considerably by adding NaOH so as to maintain this alkalinity.

As for the effect of alkalinity on the nitrification rate, a high rate of nitrification can be ensured at an alkalinity of more than 160 as shown in Fig. 9.

It is to be noted, however, that if alkalinity declines below 100, pH could decrease and the nitrification rate also becomes instable.

Fig.-8 Relation between Alkalinity and pH

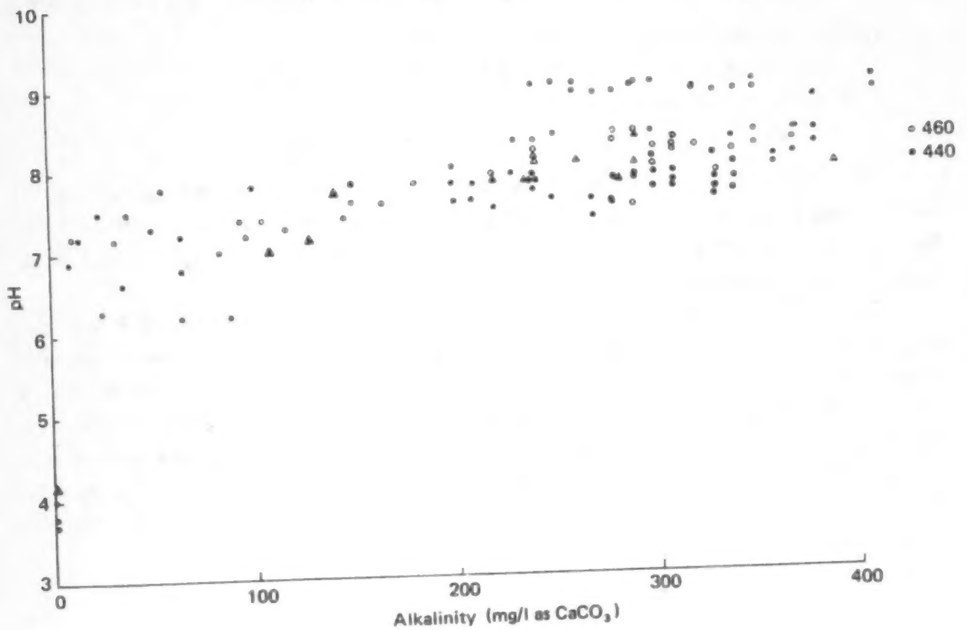
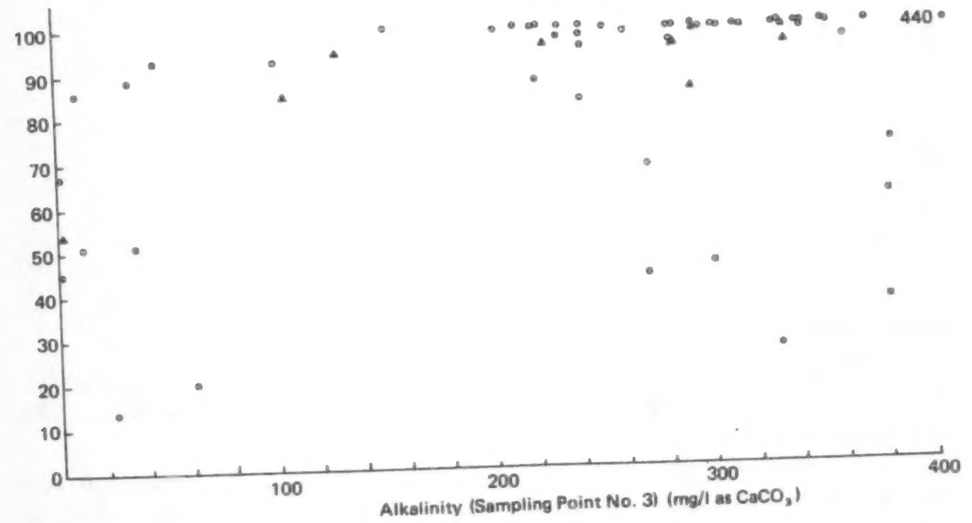


Fig.-9 Effect of Alkalinity on Nitrification Rate



3.3.5 Total Sludge Production

As sludge production data before pH control were limited and sludge formation was subject to a large fluctuation, the total sludge production in the plant was obtained by summing up the sludge volumes recorded from April 1976 to March 1977.

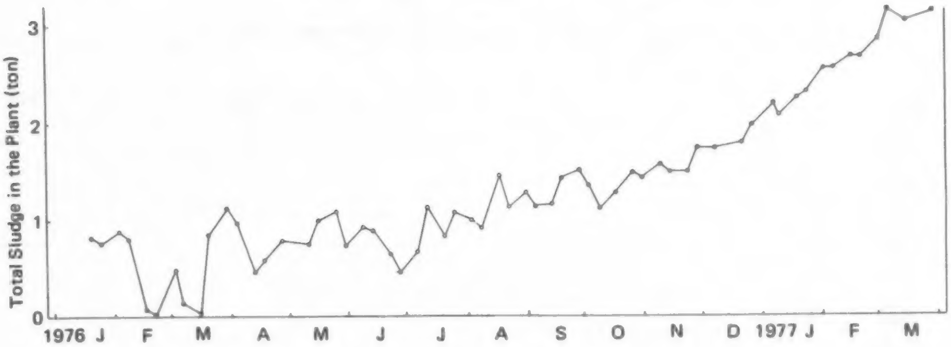
As can be seen in Fig. 10, sludge production through the year depicts two patterns. Specifically, the volume of sludge varies by temperature as shown below.

High temperature season – May ~ October:
(Water temperature – 15 ~ 20°C) SS – 4.3 kg/day
Low temperature season – November ~ April:
(Water temperature – 8 ~ 15°C) SS – 13.6 kg/day

The coefficient of correlation between sludge production and season was 0.6 for the high temperature season and 0.9 for the low temperature season. Sludge production in the low temperature season was about three time as large as that in the high temperature season.

The increase of MLSS in the aeration tank cannot be estimated with accuracy only on the basis of sludge production because the respiration and fauna of activated sluge works as a contributing factor. However, calculation was conducted on the basis of SS increase and tank capacity, which disclosed that the daily increment of MLSS is 12 mg/l in the high temperature season and 37 mg/l in the low temperature season. Hence, it can be concluded that the monthly increment of MLSS is about 400 mg/l in the high temperature season and about 1,000 mg/l in the low temperature season.

Fig.-10 Seasonal Variation of Total Sludge



3.3.6 SRT

SRT is one of the important incentive factors to maintaining substantial nitrification of activated sludge process. It is generally accepted that the longer SRT, the higher the activity can be maintained.

Data of weekly survey indicates that SRT increased from about 45 days in March (before pH control) to 230 ~ 270 days in the May ~ September period, recording about 80 days in the subsequent low temperature season, which is long enough for the growth of nitrifying bacteria.

Month	Mar.	May	Jul.	Sept.	Nov.	Feb.
SRT (days)	45.7	272	255	227	79.3	80.4

$$\text{SRT} = \frac{\text{Total volume of activated sludge in the plant}}{\text{Discharged SS + Waste activated sludge}}$$

In order to maintain nitrification when the water temperature is about 10°C, more than 12 days of SRT is reported to be required. From the data given above, therefore, it can be concluded that year-round nitrification can be expected of the plant.

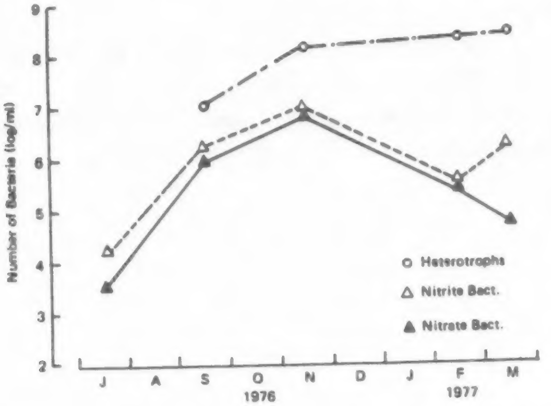
3.3.7 Micro Fauna

The fauna of activated sludge is composed of bacteria, Protozoa and Metazoa. Protozoa is classified into swim t., worm t. and fixed 6., the latter two dominating when activated sludge is in a condition favourable for their propagation. When extended aeration is carried out and sludge has high activity of nitrification, on the other hand, Metazoa and Euglypha of Protozoa prevail. Swim t. dominates when the organic matter loading is high or the water temperature is low.

The survey revealed that Euglypha, Vorticella, Litanotus and Rotifera were found relatively frequently through the year. It is known that Vorticella and Litanotus emerge in the normal activated sludge, and Euglypha and Rotifera when SRT is long (in extended aeration). In the aerobic digestion tank, it was found that Englypha was dominant. The frequent occurrence of large numbers of Euglypha and Rotifera is indicative of the probability that SRT is longer than 20 days.

As for bacteria, those working for removal of organic matters (about 10^8 N/ml) and nitrobacteria (about $10^{6 \sim 7}$ N/ml) were identified, which suggests that the bacterial activity is maintained at a high level both for removal of organic matters and for nitrification (See Fig. 11).

Fig.-11 Seasonal Variation of Bacteria in Aeration Tank



Installation of filters is very important especially at a plant expecting nitrification. The flocs of activated sludge entrapped in the filter contained lots of microorganisms. Biological examination in November disclosed that these microorganisms included 73,000 N/ml of Euglypha and 18,000 N/ml of mastigopha, the concentration of the former being about twice as large as in the aeration tank and that of the latter about 18 times as large. About 10,000 N/ml of Rotifera was also entrapped, but the great majority was found dead. Other general bacteria and nitrobacteria were found alive in the flocs in a quantity of about $10^{7 \sim 8}$ N/ml.

Backwash waste contains a far less quantity of microorganisms than in the aeration tank as it is returned only about once in 48 hours, so that it may be contributing to the bacterial activity in the aeration tank only to a small degree. However, it is much more meaningful to return the backwash waste to the aeration tank than to discharge it outside the nitrification system because of its rich bacterial activity mentioned above.

3.3.8 Consumption of NaOH

NaOH consumption was estimated from the raw sewage flow, and from alkalinity, TKN and pH of influent.

NaOH wad added at a dosage rate of 170 ml/min of 25% solution. With a daily NaOH consumption of about 61 kg, pH was maintained at 8 ~ 8.5.

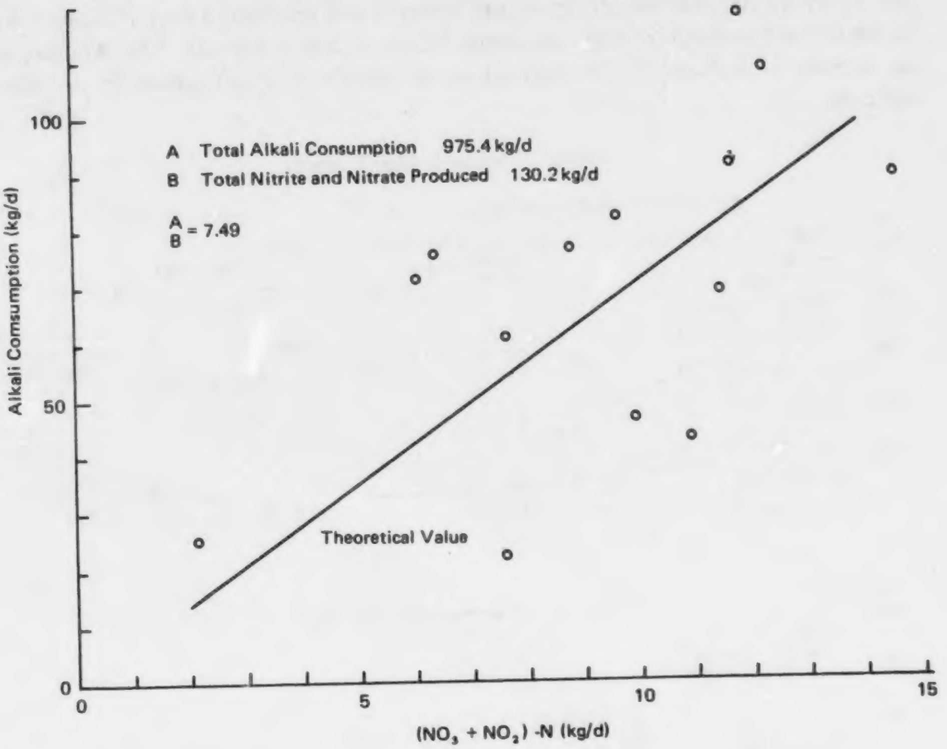
Influent TKN as revealed by the survey ranged from 7 to 16 kg·N/day, averaging 9.8 kg/day, The NaOH requirement for nitrifying this inflow TKN completely turned out to be 56 kg/day. As it was confirmed that nearly 100% of influent TKN was nitrified, the estimated dosage of 61 kg/day can be considered to be adequate.

After pH control was effected, secondary effluent presented a residual alkalinity of 100 ~ 390 as $CaCO_3$ mg/l. This points to the need for controlling NaOH desage by checking the residual alkalinity in order to reduce overall NaOH consumption.

Although the optimal pH for the activity of nitrobacteria is 8 ~ 8.5, it is known that nitrifer in activated sludge exhibits the same high activity at pH 7 ~ 9. Hence, it is considered possible to maintain a high nitrification rate even if the plant is run with pH value held at 7.5 ~ 8.

Fig. 12 shows the relationship between $(NO_3 + NO_2)$-N and alkali consumption. Oxidation of 1 mg of NH_4-N calls for the input of 7.1 mg of alkali theoretically, but the actual alkali consumption is usually smaller than this theoretical value. In this case of the plant, however, alkali consumption of 7.5 mg.

Fig.-12 Relation between $(NO_3 + NO_2)$-N and Alkali Consumption



3.3.9 Water Balance

In order to clarify the water balance at the plant, the monthly values of total tap water consumption, restaurant water consumption, dilution water volume, total reuse water volume and inflow volume were obtained to determine the average daily value for each, and the total reuse volume was divided into dilution water volume and final effluent volume by applying the mixing ratio of dilution water and inflow.

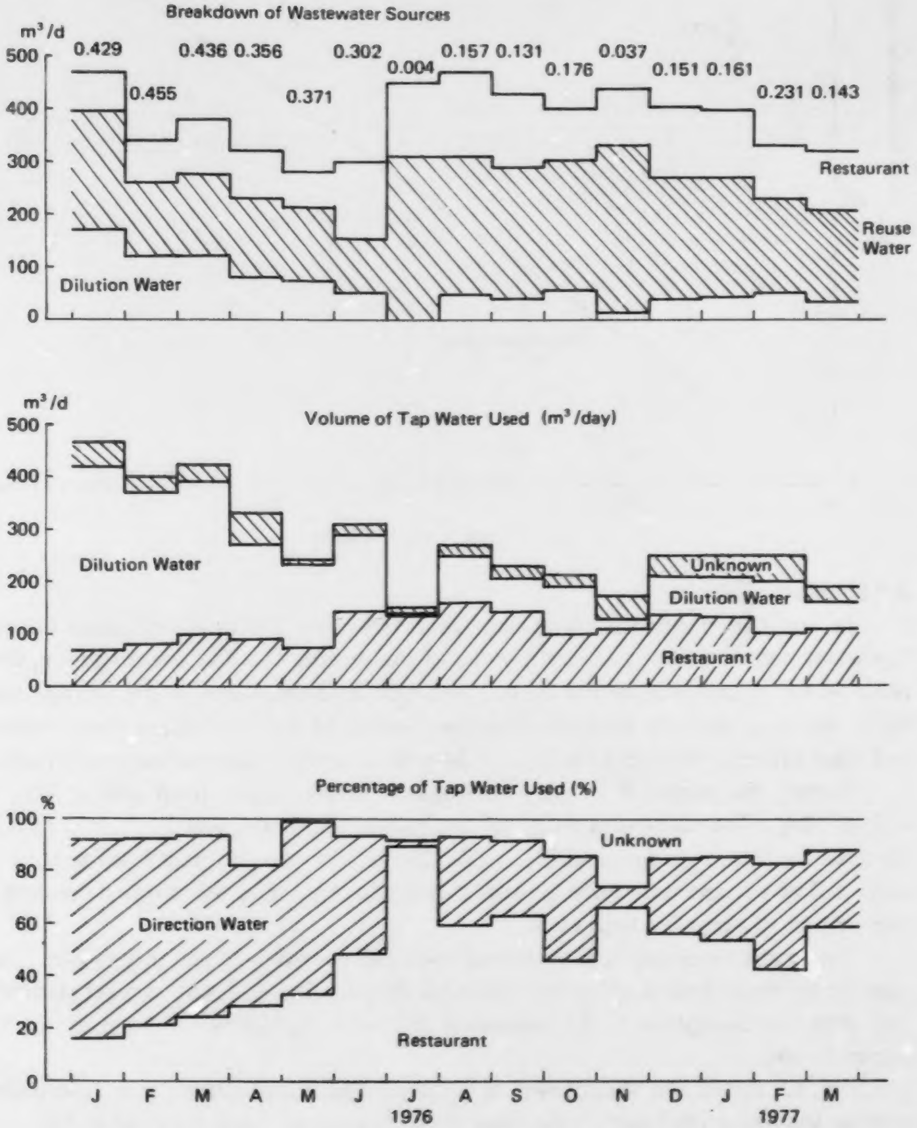
During the period from May to August, inflow ranged from 280 m^3/day to 470 m^3/day. The maximum daily inflow, which is not confirmed yet, will have to clarified by future studies as it is possible that inflow into the plant is influenced by daily inflow volume especially in August and January when the residual concentration of NH_4-N records a high value.

Tap water consumption decreased after pH control was put in practice. This was due to the dwindled use of tap water for diluting final effluent to meet standard. Tap water consumption at the restaurant showed a slight upward trend during the survey period.

Fig. 13 shows the breakdown of influent. As seen in this figure, the inflow volume showed a tendency to decrease from January to June, increase in July, and drop again from August to March. From summer to autumn, the inflow was relatively large, registering about 400 ~ 450 m^3/day, which is likely to be ascribable to the increased inflow of reuse water.

The share of reuse water in the total inflow started rising from about July when the water quality was stabilized by pH control, and recorded about 60% from July to March, accompanied by the increased inflow of dilution water. This will give rise to augmented volume and concentration of various salts and nitrate in circulating effluent.

Fig.-13 Wastewater Mass Balance



Breakdown of Wastewater Sources

Volume of Tap Water Used (m³/day)

Percentage of Tap Water Used (%)

3.3.10 Nitrate in Reuse Water and Influent

After effecting pH control, tap water consumption for diluting effluent declined because the effluent transparency increased to more than 100 and its NH_4-N concentration became as low as less than 1 mg/l. As a consequence, effluent reuse rate became higher, with the result that NO_3-N concentration showed a tendency to increase both in influent and effluent. Specifically, NO_3-N concentration in final effluent which was in the neighbourhood of 22 mg/l before pH control increased to 40 ~ 100 mg/l. This growing tendency of nitrate nitrogen concentration will pose itself as a serious problem if restriction is placed on total nitrogen concentration at some future date.

Fig. 14 shows the relationship between NO_3-N flowing into the plant in reuse water and that carried in the inflow sewage. NO_3-N concentration in the influent kept on rising from June to November, and it is likely that this tendency will continue in future. The concentration dropped in December but the reason for this is not known. The concentration in reuse water is far higher than that in influent. The shaded section in Fig. 14 indicates the unknown portion. Assuming that this section represents the portion denitrified between the reuse water inlet and the balancing tank outlet, calculation of the denitrification rate produces the results shown in Fig. 15 which indicates about 50% denitrification rate for the period from April to September and 30 ~ 40% rate from October to December. The figure shows that denitrification did not take place only for a few months in winter.

While the location of the exact denitrifying point calls for an another survey, it is possible that denitrification has taken place in the inflow pit. This is turn calls for further studies to be made to clarify the NH_3-N balance.

Fig.-14 Change in Nitrate and Nitrite (Influent Nitrogen vs. Renovated Water Nitrogen)

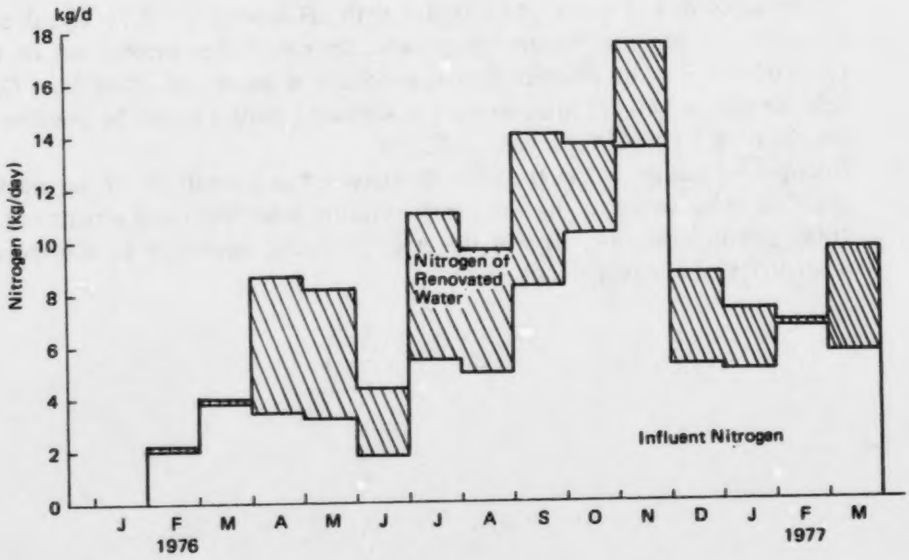
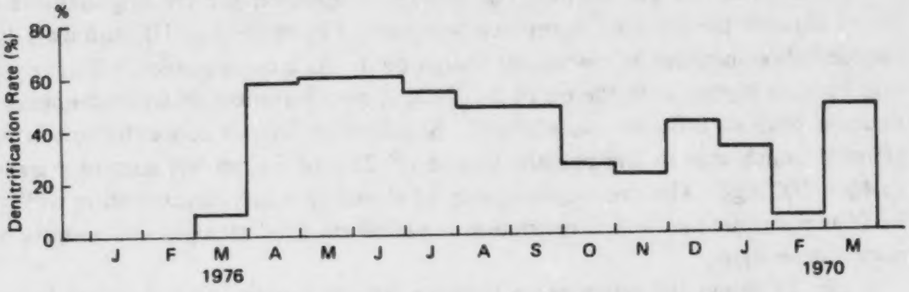


Fig.-15 Denitrification Rate



4. SUMMARY

By conducting pH control at the plant, the following facts were made clear.

1) Flushing water for water closet can be supplied without diluting effluent with tap water.

2) Nearly 100% nitrification rate can be maintained through the year. According to the weekly survey, however, a high ammonia concentration was detected due to retarded nitrification in May, August and January when the service area is utilized by many people. It may become necessary to resume the operation of nitrification tanks depending on the cause of this high residual ammonia concentration which needs to be cleared up by another aurvey.

3) Daily sludge production is estimated to be 10 ~ 40 mg/l. Since the plant is provided with a filters, frequent sludge with drawal is not considered essential. Further, the long SRT will make it possible to hold the nitrification activity at a high level through the year.

4) NaOH Consumption
NaOH was dosed at a rate of 61 kg/day with pH held at 8 ~ 8.5. This dosage proved to be generally acceptable because the calculation worked out on the basis of the weekly 24-hour survey produced a dosage of 56 kg/day. Considering the activity of nitrobacteria in activated sludge, it will be possible to run the plant with pH value held at 7.5 ~ 8.

5) Nitrogen concentration in the effluent increased as a result of the augmented share of reuse water in the total inflow (more tahn 50%) and attainment of 100% nitrification rate. It will therefore be found necessary to incorporate denitrification at a future date.

CHAPTER 2

I. PRACTICE OF NITROGEN REMOVAL BY BREAKPOINT CHLORINATION PROCESS

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1. METHOD

Pilot plant tests for nitrogen removal by breakpoint chlorination (BPC) process were performed from October 1977 to March 1978 at Minamitama Municipal Sewage Treatment Plant in Tokyo. The effluent from the tertiary treatment facilities consisting of chemical coagulation and sand filtration process was used for the pilot plant study. A schematic flow diagram of the pilot plant and the dimension of each unit are shown in Fig. 1 and Table 1 respectively. The influent NH_3-N concentration examined in this study was two concentration levels of 6 mg/l and 20 mg/l. Ammonium sulfate was added into the ammonia dosing tank for adjusting the influent NH_3-N concentration for this breakpoint chlorination study. Also, caustic soda was added to the No. 1 pH adjusting tank for a pH control in which a cascade control system with a connection of a pH meter in the following chlorine dosing tank was employed. The effects of pH were studied by varing the pH value of the sample wastewater in the chlorine dosing tank at pH 6, 7 and 8. Chlorine dosing rate was automatically controled with a NH_3-N meter (TECHNICON MONITOR IV) and a ratio slider. The ratio of chlorine dosage to nitrogen content was varied in a range from 4 to 16 and the weight of a chlorine gas cylinder (capacity, 1 ton) was, further, measured to ascertain the Cl_2 : NH_3-N weight ratio.

Three types of activated carbon originated from coconut shell (granular, crushed), spherical coal (granular) and crushed coal (granular) were evaluated by installing three carbon adsorbers in parallel. Flow rate through the carbon bed was always automatically adjusted to be 45 m^3/hr. Except Staturday and Sunday, the pilot plant was continuously operated in a base of 24 hours per day. One operational condition was continued at least for two day.

Fig. -1 Schematic Flow Diagram of the Pilot Plant

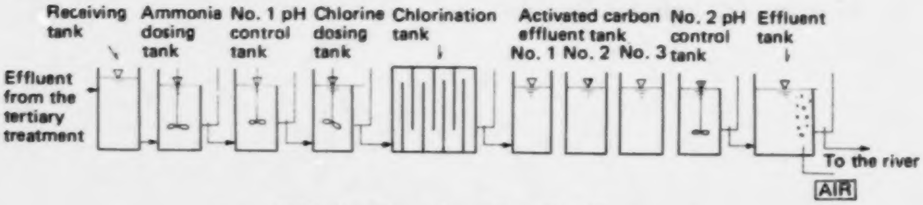


Table-1 Dimensions of the Test Plant

Equipment	Quantity	Dimensions (Effective Height) WxLxH, m	Capacity m^3
Receiving tank	1	3 x 5.5 x 3.5 (2.9)	48.0
Ammonia dosing tank	1	2.43ϕ x 3.07 (2.48)	11.5
No. 1 pH control tank	1	2.43ϕ x 3.07 (2.48)	11.5
Chlorine dosing tank	1	1.45ϕ x 1.85 (1.51)	2.5
Chlorination tank	1	6 x 7 x 2.5 (2.20)	92.0
Activated carbon adsorber	2	2.5 x 2.5 x 7.00 (6.00)	37.5
Activated carbon adsorber	1	2.7 x 2.7 x 7.00 (6.00)	43.7

Table-1 Dimensions of the Test Plant (Continued)

Equipment	Quantity	Dimensions (Effective Height) W x L x H, m	Capacity m^3
Activated carbon effluent tank	3	2.03ϕ x 3.05 (2.47)	8.0
No. 2 pH control tank	1	2.43ϕ x 3.07 (2.48)	11.5
Effluent tank	1	2.5 x 10 x 2.50 (2.00)	50.0
Ammonium sulfate storage tank	1	1.15ϕ x 2.35	1.2
Ammonium sulfate dissolution tank	1	1.54ϕ x 1.84 (1.40)	2.6
Ammonium sulfate dosing head tank	1	0.80ϕ x 1.40 (1.10)	0.6
Caustic soda storage tank	2	1.94ϕ x 2.45 (2.20)	6.5
Caustic soda dosing head tank	1	0.8ϕ x 1.40 (0.99)	0.5
Caustic soda dosing head tank	1	1.15ϕ x 1.40 (0.96)	1.0
Activated carbon storage tank	1	2.20 x 4.00 x 3.45	16.0

2. CHLORINE REQUIREMENT

Relationships between Cl_2 : NH_3-N weight ratio (R) and ammonia nitrogen removal rate are shown in Fig. 2.1 and 2.2. The concentration of ammonia nitrogen of the sample wastewater in the ammonia dosing tank and the chlorine dosing tank was measured to obtain the ammonia nitrogen removal rate. The each average value in continuous operational condition is plotted in these figures. Caustic soda was

Fig. -2.1 Ammonia Removal by Chlorination

NH_3-N Average Concentration 5.9 mg/l
pH Adjusted at 6, 7 and 8
(Figures Denote the Adjusted pH Value)

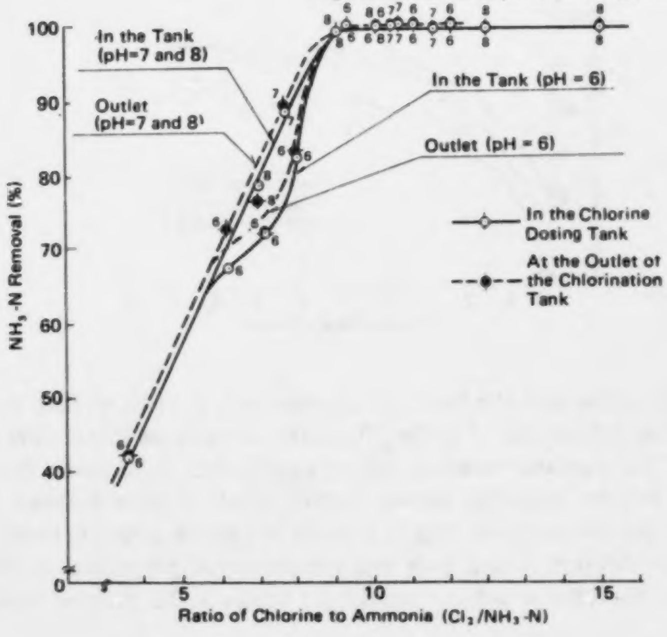


Fig. -2.2 Ammonia Removal by Chlorination

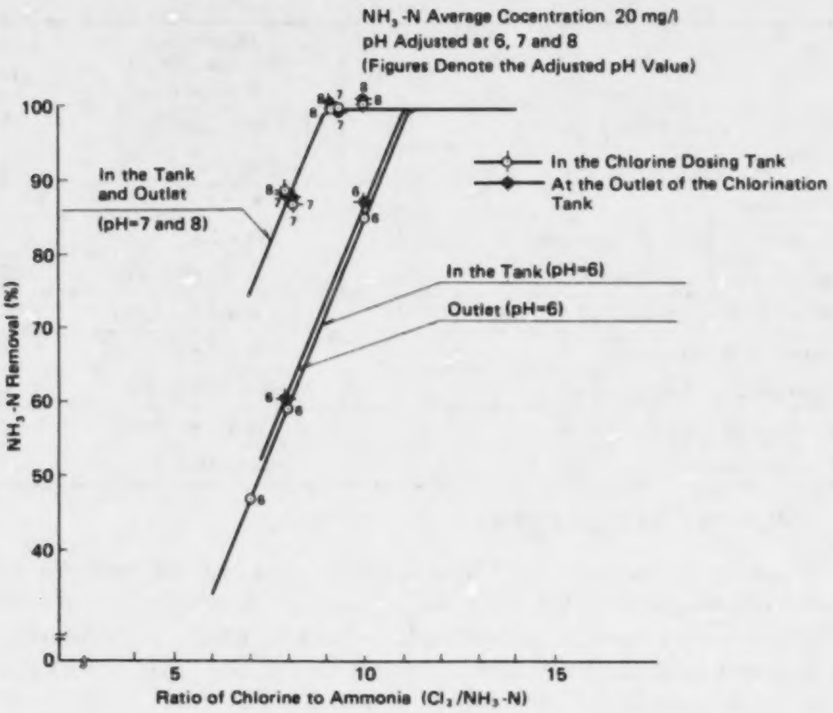
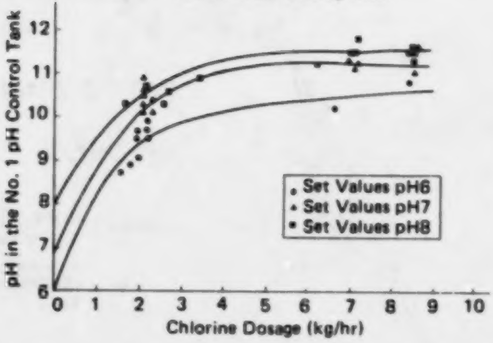

NH$_3$-N Average Cocentration 20 mg/l
pH Adjusted at 6, 7 and 8
(Figures Denote the Adjusted pH Value)

Fig. -2.3 Relationship between the set pH Values and Measured pH Values in the Chlorine Dosing Tank



automatically added into the No. 1 pH control tank in order to keep a set pH value in the chlorine dosing tank. For the pH control, a feedback system was used, which consisted of the regulator receiving source signals from a pH meter in the chlorine dosing tank and the regulator sending control signals to valve dosing a caustic soda to the No. 1 pH control tank. Fig. 2.3 shows the relationships between the set pH values in the chlorine dosing tank and the measured pH values in the No. 1 pH control tank. In order to achieve the set pH values in the chlorine dosing tank, the pH value in the No. 1 pH control tank has to be raised previously as shown in Fig. 2.3. In the case of the low concentration of ammonia nitrogen in the sample wastewater, the almost complete ammonia nitrogen removal in the chlorine dosing tank can be achieved at Cl_2 : NH_3-N weight ratio 9.5. The above results were observed at every set pH value (6 ~ 8) in the chlorine dosing tank. At the lower Cl_2 : NH_3-N weight ratio in a range from 6.0 to 8.0, however, the ammonia nitrogen removal was slightly higher at pH 6 than at pH 7 and 8.

In the case of the high concentration of ammonia nitrogen in the sample wastewater, the ammonia nitrogen removal in the chlorine dosing tank at pH 6 was significantly different from it at pH 7 and 8. At pH values of 7 and 8, the chlorine dosage at Cl_2 : NH_3-N ratio 9.5 was enough to accomplish the almost complete ammonia nitrogen removal. But the Cl_2 : NH_3-N ratio 11.5 was necessary at pH 6.

No further increase of the ammonia nitrogen removal was observed at the chlorination tank outlet. So it could be said that the reaction of chlorine and ammonia nitrogen was already completed in the chlorine dosing tank.

Regarding the pH values in the No. 1 pH control tank, they are increased to pH 9.5, 10.0 and 10.5 at the lower chlorine dosing ratio and to pH 10.5 at the lower chlorine dosing ratio and to pH 10.5, 11.25 and 11.50 at the higher chlorine dosing ratio in order to achieve the set pH values 6, 7 and 8 respectively in the chlorine dosing tank.

The activated carbon adsorption unit following the chlorination tank has increased the ammonia nitrogen removal ratio as shown in Fig. 2.4 through 2.6. At the low concentration of nitrogen of the influent and at the low ammonia nitrogen removal ratio in the BPC process, the additional activated carbon treatment increased the ammonia nitrogen removal rate by 10 ~ 30% (Fig. 2.4). The ammonia nitrogen removal effect, however, decreases when achieving the removal rate higher than 90 ~ 95%. In order to accomplish 100% ammonia nitrogen removal, about 9.5 in Cl_2 : NH_3-N ratio is necessary. Then, it means that the required Cl_2 : NH_3-N ratio is same as it in the case of no additional carbon unit.

When the requirement of the ammonia nitrogen removal is assumed to be 95%, the addition of the activated carbon unit can decrease the Cl_2 : NH_3-N weight ratio in the BPC unit from 8 to 7. At the high concentration of ammonia nitrogen in the influent, the pH value has an influence to the ammonia nitrogen removal effect of the activated carbon. As shown in Fig. 2.5, about 30% increase in the ammonia nitrogen removal rate observed resulting in a decrease of chlorine dosing ratio from 11.5 to 10 on the complete ammonia nitrogen removal. The increase of the ammonia nitrogen removal rate by the activated carbon unit was only 10% at pH 7 and 8 and the chlorine requirement could not be decreased in the complete ammonia nitrogen removal. In the case of the ammonia nitrogen removal rate of 95%, the additional carbon unit decreased the chlorine requirement from Cl_2 : NH_3-N ratio 8.8 to 8.3. For the 100% ammonia nitrogen removal and the high concentration of NH_3-N in the influent, the decrease of the chlorine requirement can not be expected even with the additional activated carbon adsorption unit.

Fig. -2.4 NH_3-N Removal by Activated Carbon

NH$_3$-N Average Concentration 5.9 mg/l
pH Adjusted at 6, 7 and 8
(Figures Denote the Adjusted pH Values)

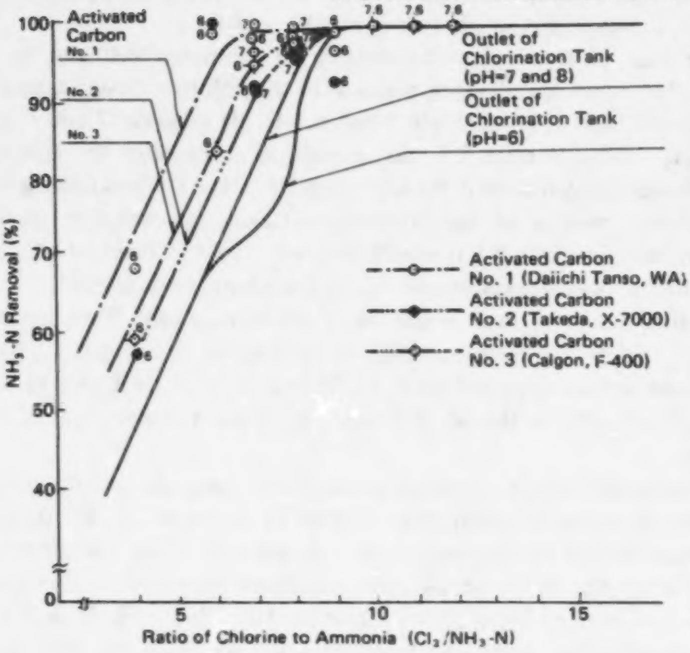


Fig. -2.5 NH_3-N Removal by Activated Carbon

Average NH$_3$-N Concentration 20 mg/l
pH Adjusted at 6

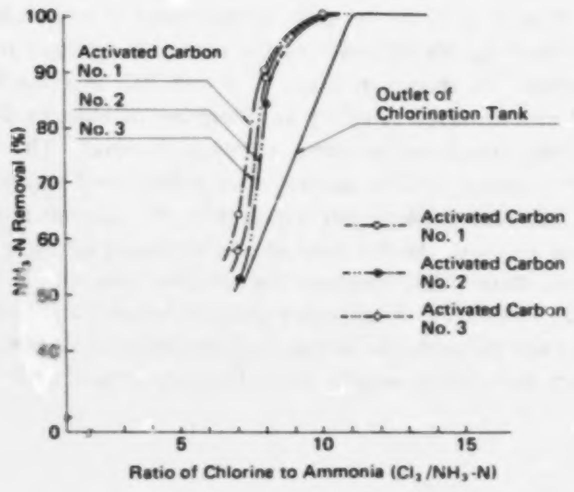
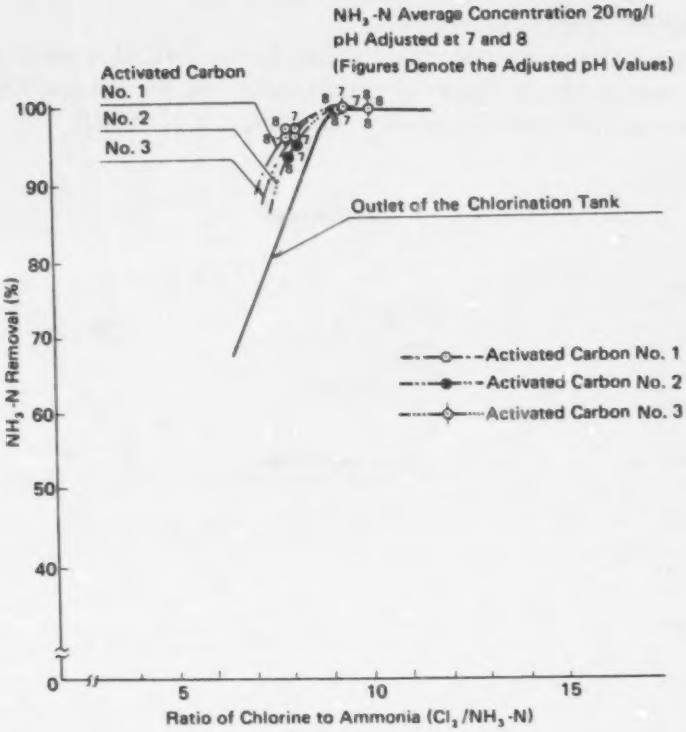


Fig. -2.6 NH_3-N Removal by Activated Carbon



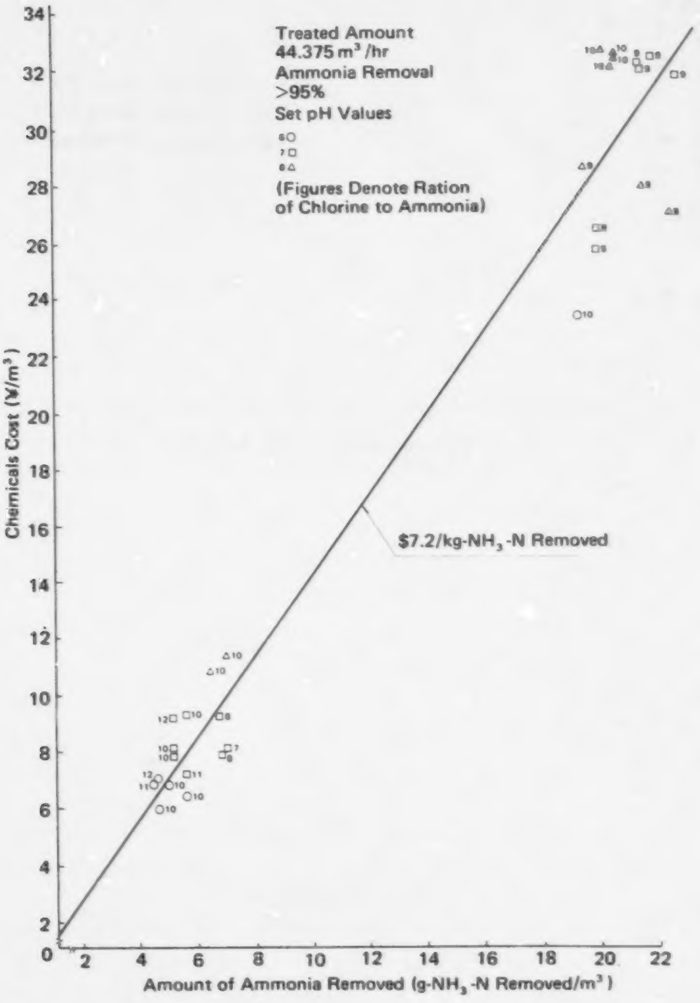
NH_3-N Average Concentration 20 mg/l
pH Adjusted at 7 and 8
(Figures Denote the Adjusted pH Values)

3. COSTS FOR CHEMICALS IN BPC PROCESS

Fig. 3.1 shows the costs of chemicals required for the nitrogen removal by the BPC process. These costs include liquid chlorine ($375/ton) and caustic soda (15% liquid $90/m³) used in pH control.

This figure indicates that the chemical cost for the NH_3-N removal is $7.2/kg NH_3-N (removed) in all the ranges of the pH values and the influent NH_3-N concentrations studied in this pilot plant tests. (Changing rate $1=¥200)

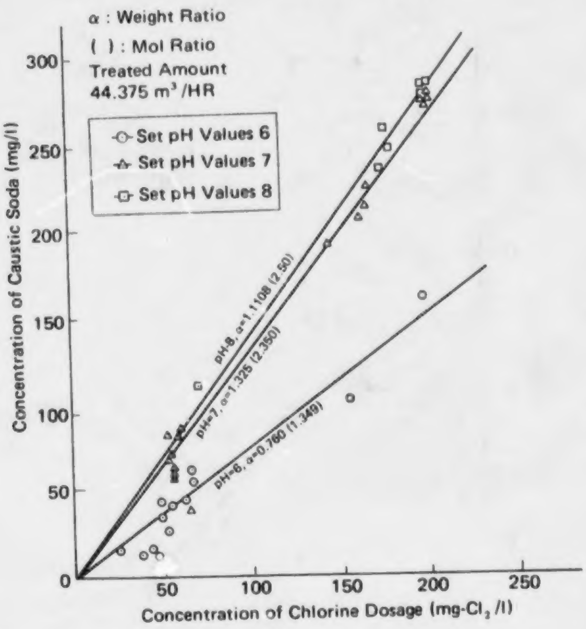
Fig. -3.1 Chemicals Costs



4. CAUSTIC SODA CONSUMPTION

Caustic soda requirement to control the set pH values for the breakpoint reaction is shown in Fig. 4.1. Mol (weight) ratio of the required caustic soda to the amount of chlorine added is 1.35 (weight ratio 0.76) at the set pH values of 6, and also 2.35 (1.33) and 2.50 (1.41) at the set pH values of 7 and 8 respectively.

Fig. -4.1 Consumption of Caustic Soda



The overall reaction of the breakpoint chlorination is represented by the equation (1). The reaction presented by the equation (2) shows the neutralization reaction.

$$2NH_3 + 3Cl_2 \rightarrow 2N_2 + 6HCl \qquad (1)$$

$$6HCl + 6NaOH \rightarrow 6NaCl + 6H_2O \qquad (2)$$

According to the above equations, the mol ratio of caustic soda required to chlorine added to adjust at the pH value of 7 is 2.0. In this test, it resulted in 2.35. It is, therefore, considered that the sample wastewater has the buffer effects to the pH elevation at pH around 7.

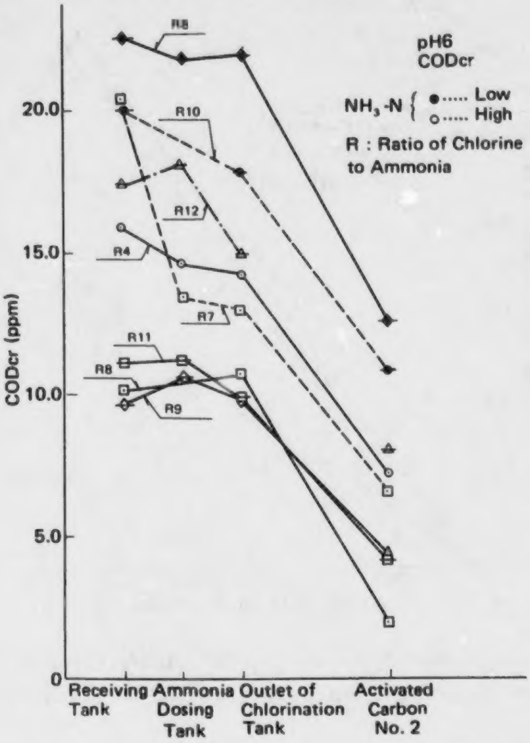
5. CHANGES OF WATER QUALITY

Water quality changes at various points through the treatment system, such as at the receiving tank, the chlorination tank outlet and the activated carbon adsorber (No. 1 to No. 3) were observed.

5.1 CODcr

The changes of CODcr along the BPC process at the various Cl_2 : NH_3-N weight ratio are shown in Fig. 5.1. The initial concentration of ammonia nitrogen does not effect the CODcr behaviour. By adding chlorine, its concentration decreased in a certain degree and the further reduction of CODcr was achieved through the following activated carbon adsorber.

Fig. -5.1 Changes in CODcr Concentration



CODcr removal effects by chlorination and activated carbon adsorption are summarized in Table 2. CODcr/CODmn ratio was 3.12 which resulted from 41 samples analysis.

Table-2 CODcr Removal Rate (%)

Influent concentration of NH_3-N	(*1) Chlorination	(*2) Activated Carbon Adsorption		
		No.1	No.2	No.3
Low concentration (about 6 mg/l)	11.8	37	49	46
High concentration (about 20 mg/l)	5.8	33	43	42

$$*1 = \frac{A - B}{A} \times 100 \qquad *2 = \frac{B - C}{B} \times 100$$

A (mg/l) : CODcr (mg/l) at the ammonia dosing tank

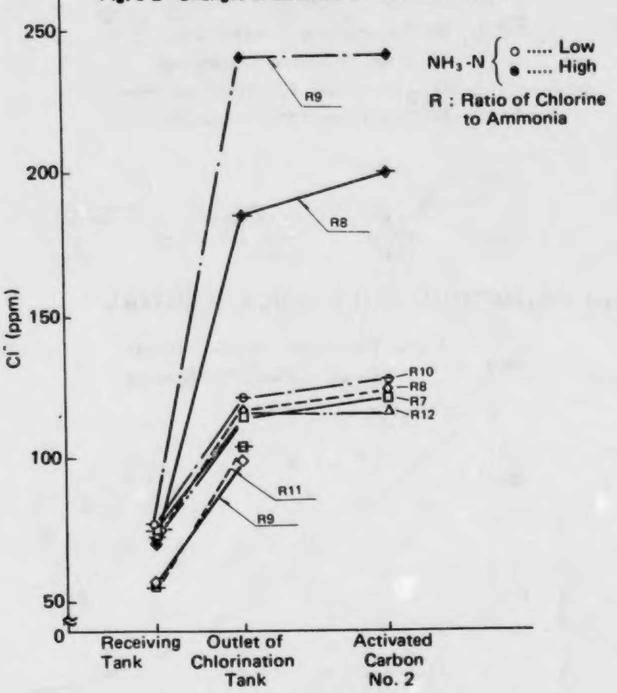
B (mg/l) : CODcr (mg/l) at the chlorination tank outlet

C (mg/l) : CODcr (mg/l) at the carbon absorber outlet

5.2 CHLORIDE ION

The changes of chloride ion at the set pH value of 7 are shown in Fig. 5.2. A concentration of Cl^- in sample wastewater (secondary effluent) was in a range of 60 and 90 mg/l. The decomposition of added chlorine increases chloride in the effluent as shown in Fig. 5.2. The concentration of chloride, further, increased at the activated carbon adsorber outlet. This implies that the decomposition of residual chlorine occurs in the carbon bed.

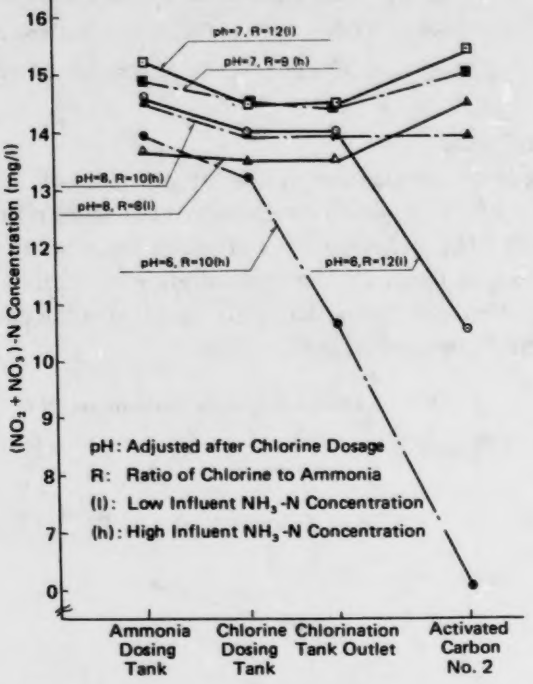
Fig. 5-2 Changes in Chloride Concentration pH 7



5.3 NITRATE AND NITRITE NITROGEN

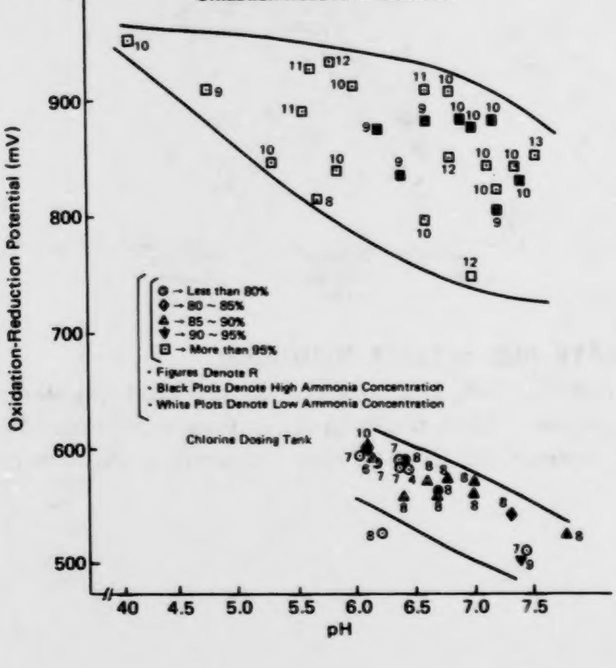
At a high Cl_2 : NH_3-N ratio in the BPC process, hypochlorous acid may oxidize ammonia nitrogen, which results in the increase of nitrate and nitrite nitrogen. In this study, however, this increase was not observed as shown in Fig. 5.3.

Fig. -5.3 Changes in (NO_2+NO_3)-N Concentration



6. pH AND OXIDATION-REDUCTION POTENTIAL

Fig. -6 Relationship between pH and Oxidation-Reduction Potential



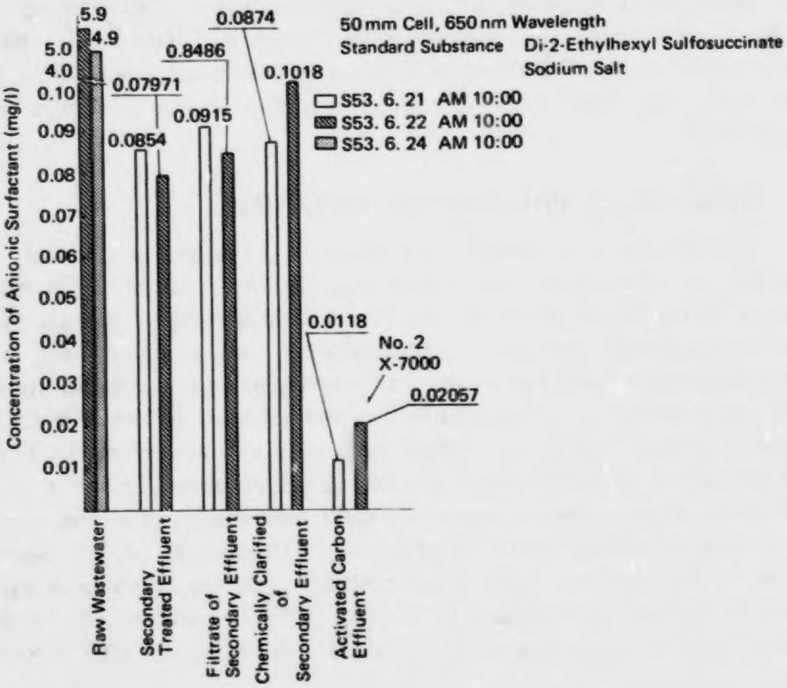
Oxidantion-reduction potential value of the solution will increase suddenly when the reaction reaching at the breakpoint. As shown in Fig. 6, the value in less than 600 mV before reaching the breakpoint, that is in the lower ammonia nitrogen removal zone of the breakpoint curve, even at a low pH value of wastewater.

In the zone that Cl_2 and NH_3-N reaction is near the breakpoint and the ammonia nitrogen removal rate is greater than 95%, the potential value increases suddenly. The potential values at pH 6.0 and 7.0 are in the ranges from 800 to 950 mV and from 750 to 900 mV respectively. This indicates that the lower the pH value, the higher the oxidation-reduction potential value. It also can be said that the breakpoint reaction is taking place when the potential value is higher than 700 mV.

7. REMOVAL OF ANIONIC SURFACTANTS

Raw sewage usually contains anionic surfactants in a range of 5 and 6 mg/l. This concentration of surfactants can be reduced to a range from 0.08 to 0.10 mg/l either by the secondary treatment alone, plus filtration or coagulation processes with filtration. As shown in Fig. 7, the additional activated carbon adsorption has decreased the concentration of anionic surfactant to 0.01 ~ 0.02 mg/l.

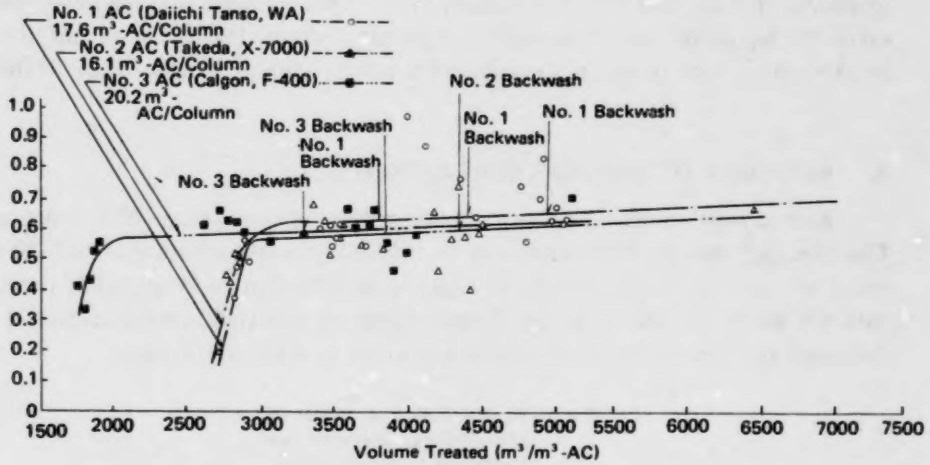
Fig. -7 Anionic Surfactant Concentrations in the Effluents of Minami-Tama Wastewater Treatment Plant



8. CODcr ADSORPTION ABILITY OF ACTIVATED CARBON

In Fig. 8, the residual CODcr in the activated carbon treatment effluent is shown. The leakage of CODcr from the carbon adsorbers is shown as a ratio of effluent concentration to the influent concentration,Ci/Co.

Fig. -8 Adsorption of CODcr by Activated Carbon

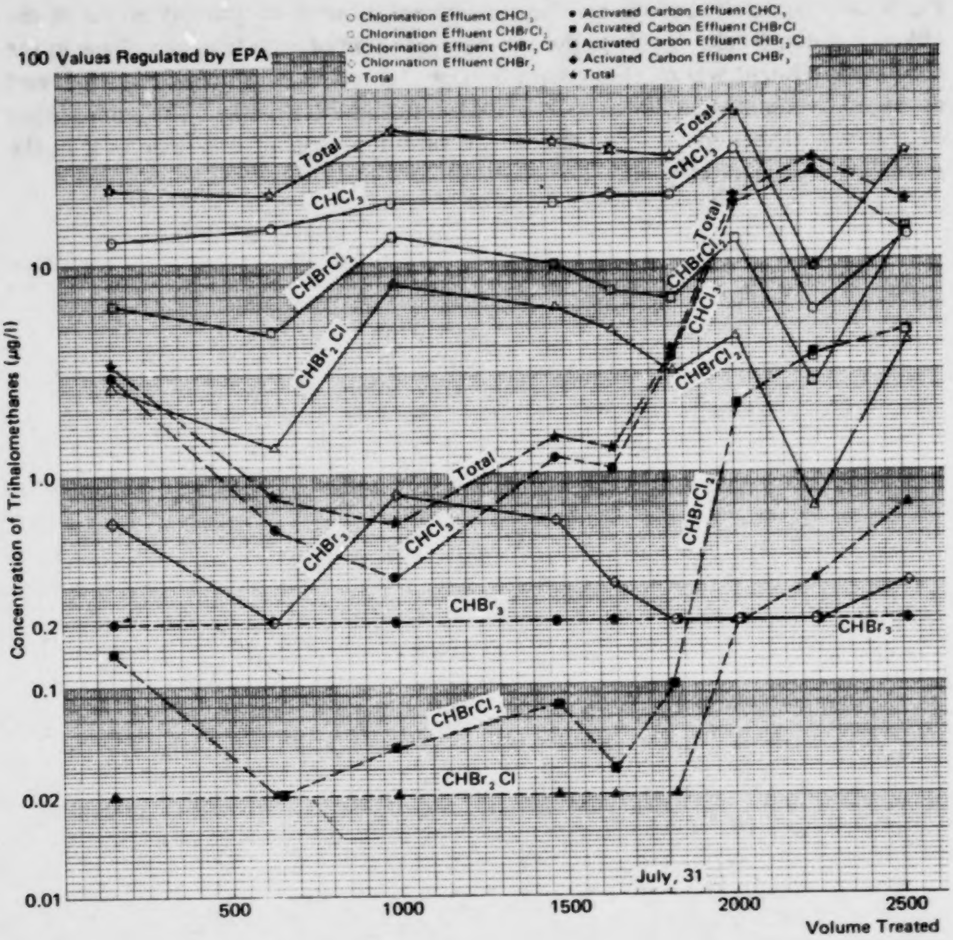


An apparent reduction of CODcr removal rate was observed when the amount of wastewater treated by activated carbon became as 1800 times as the volume of the activated carbon F-400 and 2000 times as it of the activated carbon X-7000 and WA. After then, however, these activated carbon still show the CODcr removal rate of 30 ~ 40%.

9. REMOVAL OF CHLORINATED ORGANICS

The addition of a relatively large amount of chlorine to secondary effluent in the BPC process will produce chlorinated organics. Activated carbon is able to remove them. Fig. 9 shows the changes of trihalomethanes through the activated carbon adsorption process. Trihalomethanes such as chloroform, bromoform, bromodichloromethane and dibromochloromethane can be quantitatively analysed. The concentration of trihalomethanes in the activated carbon effluent are plotted against a volume ratio of the treated wastewater volume (average 1,032 m^3/day) to the amount of activated carbon charged in the adsorber (15.6 m^3). The granular (spherical) virgin activated carbon (X-7000) was employed in this study and the operational conditions are LV (linear velocity) 6.88 m/hr and SV (space velocity) 2.8 hr^{-1}. The ammonia nitrogen concentration and the pH value of sample wastewater in the study are adjusted to 6.0 mg/l and pH 8. Before July 31, the chlorine dosing ratio is averaged to be 9.2 ($Cl_2 : NH_3$-N ratio) and after August 1, to be 11.2.

Fig. 9 Analytical Results of Trihalomethanes



As shown in Fig. 9, total trihalomethanes produced in the BPC process was 20 ~ 50 ppb, which composition is chloroform of 13 ~ 34 ppb, bromodichloromethane of 5 ~ 11 ppb, dibromochloromethane of 1 ~ 8 ppb and bromoform of less than 1 ppb. This composition was not affected by Cl_2 : NH_3-N ratio. The following activated carbon adsorption unit can reduce the concentration of total trihalomethanes to 0.5 ~ 3 ppb. Most of the residual trihalomethanes were of chloroform (0.3 ~ 3 ppb) and the others were not detected.

When the volume ratio of activated carbon treatment was increased up to 1700 ~ 1800 (m^3/m^3-AC), trihalomethanes removal effect by activated carbon decreased. Trihalomethanes removal ratio lowered to 40% when the volume ratio increased up to 2500 m^3/m^3-AC, that resulted in the residual trihalomethanes concentration of 20 ppb. This increase of the residual trihalomethanes was mostly contributed by the increase of chloroform. With respect to bromoform, no increase in its concentration was observed.

The low concentration of trihalomethanes obtained on August 10 in the Fig. 9 was probably caused by the operational trouble of the pH meter in the chlorine dosing Tank. Incidentally, the concentration of each trihalomethane in the secondary effluent was of chloroform 0.9 ppb, bromodichloromethane 0.2 ppb and dibromochloromethane 0.08 ppb. Bromoform was not detectable. The composition of trihalomethanes in the coagulation and filtration effluent was same as it in the secondary treatment effluent.

II. THE PREVENTION OF FORMATION AND REMOVAL OF CHLORINATED ORGANICS IN WASTEWATER

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II. THE PREVENTION OF FORMATION AND REMOVAL OF CHLORINATED ORGANICS IN WASTEWATER

1. INTRODUCTION

Since chlorinated organics such as chloroform had been detected in tap water and was suspected to be relevant to be carcinogenic, extraordinary attention has been paid to chlorinated organics centering on trihalomethane. Chloroform is known to be a product of reaction between chlorine and precursors, and the uncovering of the identity of precursors seems to be magnetizing the attention of those concerned. Chlorinated organics are giving rise to many concerns related to water treatment. And these problems are considered relevant to wastewater treatment as follows:

1) Chlorination disinfection is also introducted in wastewater treatment.
2) As some water treatment plants are located in the lower reaches of a wastewater treatment plant, there are possibilities of wastewater effluents being reused. In this case, there should be a possibility of precursors being contained in the wastewater effluent.
3) The breakpoint chlorination of sewage has something to do with the production of chlorinated organics.

This report deals with the above mentioned problems with special emphasis placed on 3). The breakpoint chlorination process is one of the processes of removing ammonia nitrogen, and the process is presently being experimented by the Puplic Works Research Institute, Ministry of Construction at a pilot plant in the Toba Sewage Treatment Plant in Kyoto. The pilot plant is equipped with an automatic control unit to control the ratio between ammonium and chlorine, and carbon column to remove residual chlorine. With the use of the automatic control unit, over 90 percent of ammonia nitrogen removal has been achieved. However, as there remained a certain amount of organics in the wastewater effluent, it was expected that the dosed chlorine reacted with such organics to produce chlorinated organics.

2. REACTION OF CHLORINE AND ORGANICS

2.1 INDENTIFICATION OF CHLORINATED ORGANICS

As the breakpoint chlorination process requires dosing of a large amount of chlorine, the production of chlorinated organics was anticipated. The identification of chlorinated organics was conducted by employing a Gas-Chromatograph Mass-Spectrometer (GC-MS). The identified compounds in the breakpoint chlorination effluents are listed as follows:

> carbon tetrachloride, trichloromethane (chloroform), trichloroethylenne, tetrachloroethylene, dibromomethane, dibromochloromethane, tribromomethane (bromoform), m·p-dichlorobenzene, O-dichlorobenzene, 1.2.4-trichlorobenzene and 1.2.3-trichlorobenzene.

2.2 REACTION OF CHLORINE AND NITROGEN-CONTAINING COMPOUNDS

Besides ammonia nitrogen, there are other nitrogen-containing organic compounds that consume chlorine in wastewater.

Although nitrogen-containing organic compounds or organic nitrogens are found in low concentration in the secondary effluent, they are discovered in the primary effluent at a ratio of about 1 : 1 with ammonia nitrogen. When primary effluent is to be discharged after chlorination, there is a possibility that nitrogen-containing organic compounds react with chlorine to produce organic chloramine.

In Fig. 1, sodium hypochlorite (NaOCl) was added to eight different nitrogen-containing compounds to draw chlorine dissipation curves. The compounds selected were:

aniline and cadaverine dihydrochloride as amine, N-1 naphtylacetamide and thymidine as amide, 4-aminoazobenzene as azo, histidine and tyrosine as amino-acid, and humic acid.

Fig.-1 Chlorine Dissipation Curve of Nitrogenous Compounds

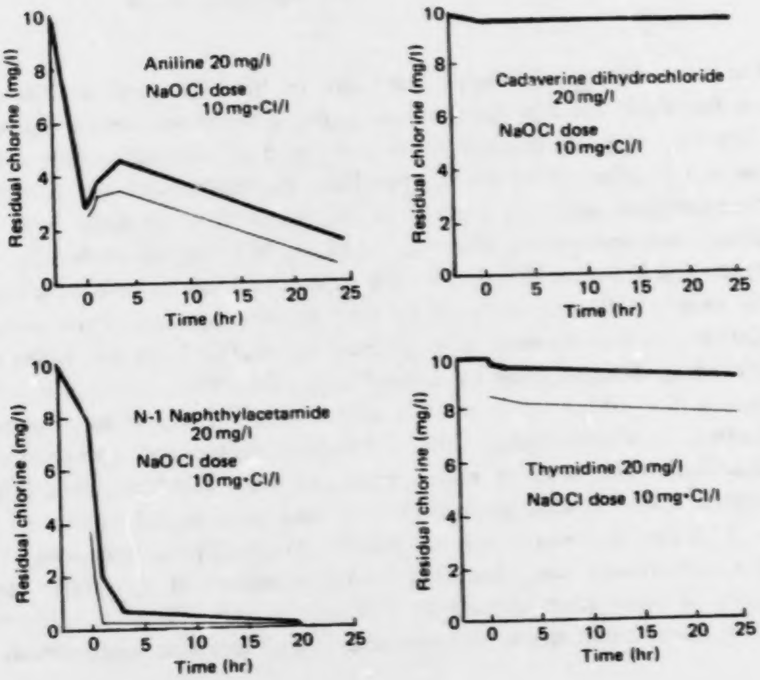
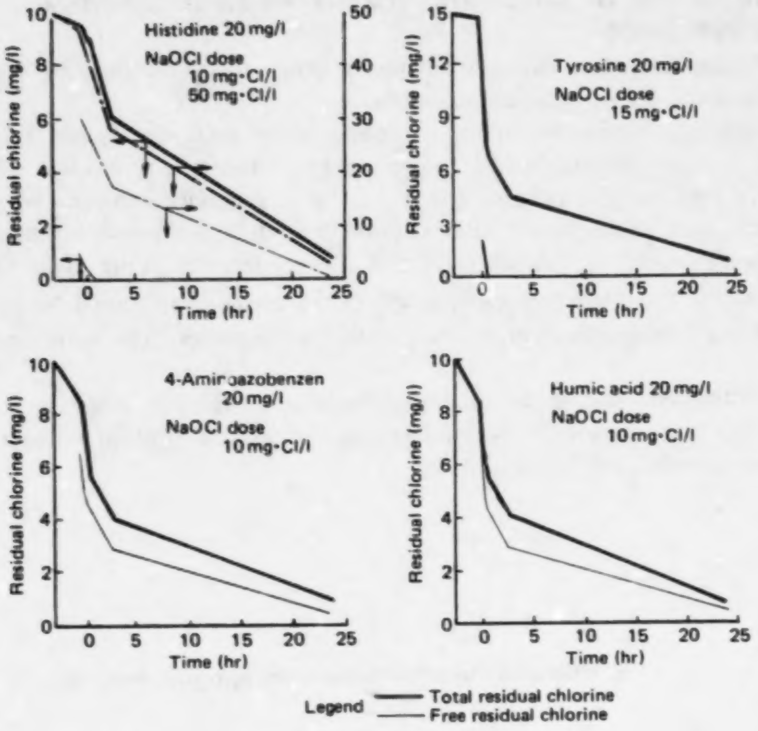


Fig.-1 (Continued)



The measurement of chlorine was based on the DPD method. The difference between the thick line and the thin line in Fig. 1 is the amount of organic chloramine formed. Organic chloramine is composed of cadaverine dihydrochloride, histidine and tyrosine. With the reaction time, the residual chlorine in histidine and tyrosine decreased and was considered to partly have changed into aldehyde. Thymidine consumed little chlorine. Aniline, N-1 naphtylacetamide, 4-aminoazobenzene and humic acid produce little organic chloramine. As these compounds have the property of aromaticity, there may be substitution reaction occured with these compounds and chlorine. The products of reaction between aniline and chlorine have partly become clear by using the GC-MS. When 10 mg·Cl/l of sodium hypochlorite was added to 20 mg/l of aniline solution, three substitutions of O-chloroaniline, m·p-chloroaniline and 2.4-dichloroaniline and non-reacted aniline were identified. The ratio of each compound was: O-chloroaniline 2.8%, m·p-chloroaniline 1.9%, 2.4-dichloroaniline 0.5% and non-reacted aniline 47.4%. All these total 52.6%. However, it was not possible to identify the remaining 47.4%.

Also, chloroform was identified in the solutions of the above mentioned compounds 24 hours after addition of chlorine. Namely, 15 μg/l of chloroform was detected in 4-aminoazobenzene solution and 60 μg/l in humic acid solution. Humic acid is known as a precursor in producing chloroform. Chloroform was not detected from other compounds.

Although the reactivity of chlorine and nitrogen-containing compounds was high, the reaction velocity was comparatively slow.

Primary effluent contained ammonia nitrogen and nitrogen-containing organic compounds. It is known that the reaction velocity between ammonia nitrogen and chlorine is fast. Accordingly, when chlorine is dosed to the primary effluent, it is considered that the chlorine first reacts with ammonia nitrogen and produces inorganic chloromine. When compared with chlorine, chloromine is reported to be inferior in reactivity. Also as the disinfection level of chlorine is only a few mg Cl/l, free chroline of high reactivity was hardly detected. From the above reasons, it was concluded that chlorine dosing on a disinfection level rarely produce chlorinated organics and organic chloramine.

3. FORMATION OF CHLORINATED ORGANICS

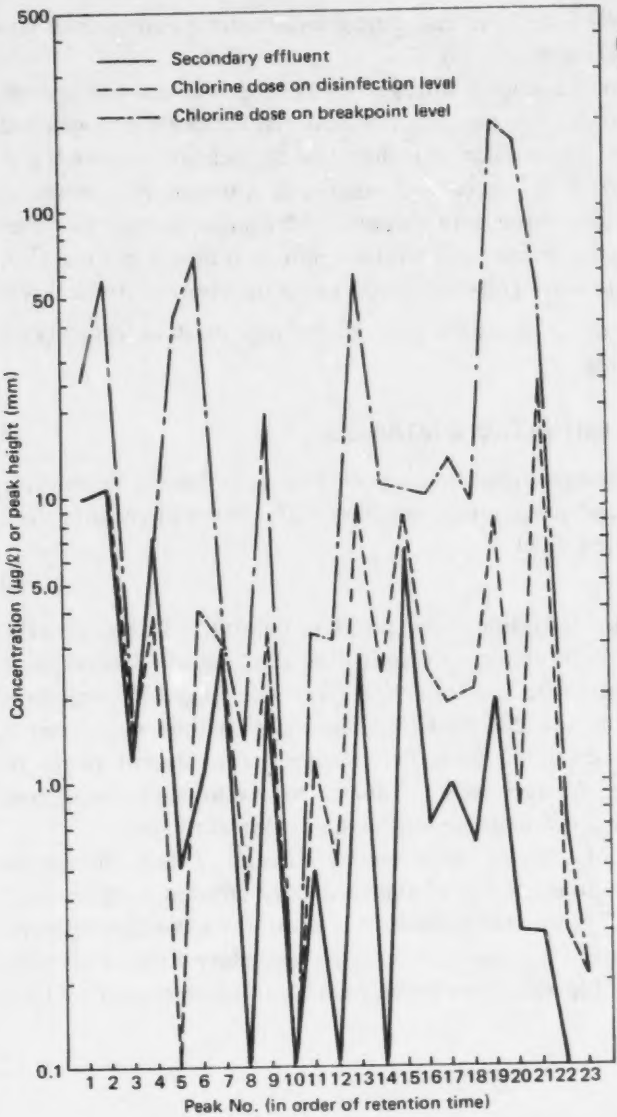
The formation of chlorinated organics is reported to be subjected to chlorine dose rate, reaction time, pH and temperature. In this study, effects of chlorine dose rate and reaction time was determined.

3-1 CHLORINE DOSE RATE

Tests were conducted to determine to what extent chlorinated organics were formed by the dosing rate of chlorine on a disinfection level and on a breakpoint level. The rate of chlorine dose on a disinfection level was 5 mg·Cl/ℓ and on a breakpoint level between 100 and 200 mg·Cl/ℓ. The reaction time was about 1 hour. The wastewater were collected from five wastewater treatment plants in Tokyo and her neighborhood. Analysis were conducted by employing GC equipped with ECD on compounds with a low boiling point such as chloroform, etc.

Fig. 2 shows the results of tests for the secondary effluent. Totally there were 23 different peaks detected, although not all the secondary effluent samples were responsible for the 23 peaks. The average number of peaks in the secondary effluent itself was 9.9. When chlorination was conducted on the secondary effluent on both the disinfection and breakpoint levels, respectively, each recorded an average of 12.5 peaks.

Fig.-2 The Effect of Chlorine Dose on Secondary Effluents



Peak No.	Compounds	(Unit)
1	CH_2Cl_2	(mm)
2	$CHCl_3$	(ppb)
3	I.I.I-Trichloroethane	(ppb)
4	CCl_4	(10^{-3} ppb)
5	Unknown	(mm)
6	Unknown	(mm)
7	C_2HCl_3	(ppb)
8	Unknown	(mm)
9	$CHBrCl_2$	(ppb)
10	CH_2Br_2	(ppb)
11	Unknown	(mm)
12	Unknown	(mm)
13	Unknown	(mm)
14	Unknown	(mm)
15	C_2Cl_4	(10^{-2} ppb)
16	$CHBr_2Cl$	(ppb)
17	Unknown	(mm)
18	Unknown	(mm)
19	Unknown	(mm)
20	Unknown	(mm)
21	$CHBr_3$	(ppb)
22	Unknown	(mm)
23	Unknown	(mm)

When comparing the secondary effluent and the disinfected effluent, except for peaks No. 13, 19 and 21, no marked differences were confirmed. The reason why No. 21 marked a great difference was perhaps due to the large value of 65 μg/ℓ detected in the effluent of one sewage treatment plant. This sewage treatment plant is located on a coastal line where its influent contained more than 2,000 mg/ℓ of chloride ion concentration due to the intrusion of sea water. Therefore, as the bromide ion concentration was higher than in ordinary wastewater, it was considered to have resulted in the more formation of bromide compounds. This, on the other hand, indicates if bromide ion exists, the reactivity between chlorine and organics becomes higher.

When dosing chlorine on a breakpoint level, it has shown a drastic difference in the height of the peaks when compared with those of the secondary effluent. Those distinct differences were in peaks No. 1, No. 2, No. 5, No. 6, No. 9, Nos. 12 ~ 14, and Nos. 16 ~ 21. Of these, the compounds identified were CH_2Cl_2, $CHCl_3$, $CHBrCl_2$, $CHBr_2Cl$, and $CHBr_3$. Although confirmed existing in the wastewater, the compounds that inevitably do not associate with chlorination are CH_3CCl_3, CCl_4, C_2HCl_3, CH_2Br_2 and C_2Cl_4.

ECD is a detector which has a peculiar sensitivity to halogenide. Also ECD's sensitivity differs greatly according to the types of compounds. By ignoring the differing sensitivity of ECD, chlorinated organics formed on both the disinfection level and the breakpoint level based on the secondary effluent were estimated from the heights of peaks. As a result, it was 1.69 times on the disinfection level and 14.8 times on the breakpoint level of the secondary effluent respectively.

In case of the primary effluent, when similar assumptions were made, it resulted in the same tendency. The results were 1.18 times on the disinfection level and 16.6 times on the breakpoint level of the primary effluent respectively.

From these results, when chlorine is dosed on a breakpoint level to the wastewater, although each chemical compounds is not identified, it has become obvious that a variety of chlorinated organics are formed in short time. Of all the identified compounds, the concentration of trihalomethane was the highest. And, if the reaction time is further lengthened , it is assumed that the concentration of trihalomethane will increase.

3.2 REACTION TIME

A large amount of chlorine is dosed to remove ammonia nitrogen by the breakpoint chlorination process. But as the breakpoint being the border, the forms of the residual chlorine differ. Inorganic chlorine exists before the breakpoint while free chlorine is found after the breakpoint. And on the breakpoint, both inorganic chlorine and free chlorine little exist. To obtain the above three conditions, chlorine was dosed to the secondary effluent. After chlorine dosing, analysis of chloroform formed were conducted at the first hour and at the 24th hour. The chloroform concentration at pre-breakpoint, breakpoint and post-breakpoint changed from 12 to 14, 39 to 43, and 88 to 260 $\mu g/1$, respectively. However, before the breakpoint and at the breakpoint, there were no marked changes in the chloroform concentration even after 24 hours. In the post-breakpoint condition, or in a condition when free chlorine exists, the chloroform concentration increased by three times with the reaction time. This indicates the fact that the secondary effluent contains a good amount of precursors that have potential to form chloroform.

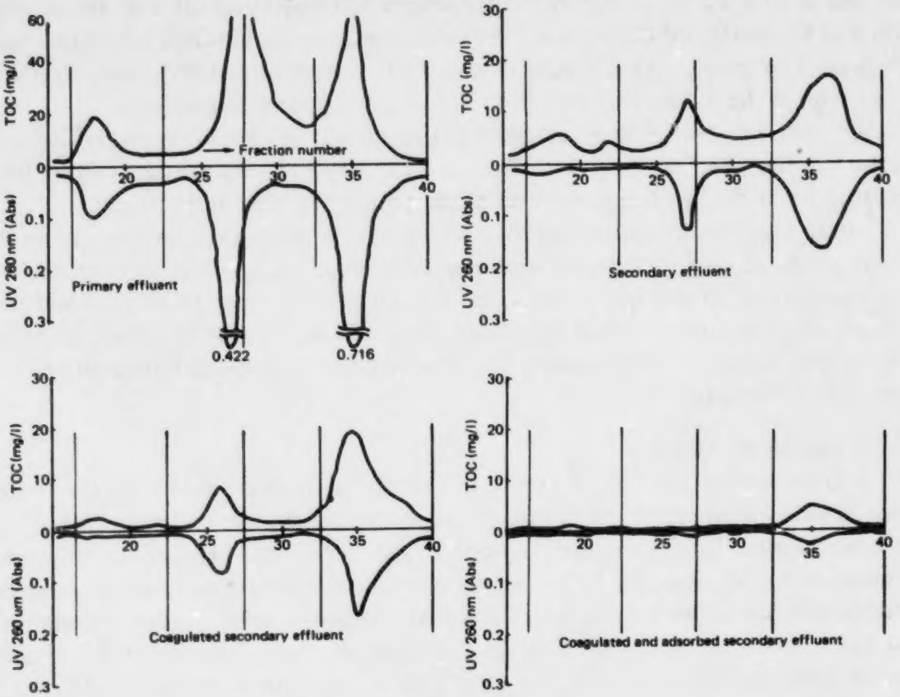
4. REMOVAL AND PREVENTION OF CHLORINATED ORGANICS FORMATION

4.1 REMOVAL PRECURSORS

4.1.1 Laboratory Tests

Gel filtration chromatography (GFC) is one type of liquid chromatography and is used as a means to evaluate the removal effects of soluble organics. Fig. 3 shows the gelchromatograms of the primary effluent, the secondary effluent, the coagulated secondary effluent and the coagulated and carbon adsorbed secondary effluent.

Fig.-3 Gel-Chromatogram of Wastewater



The gel conditions are as listed below:

gel bed	Sephadex G-15
developer	H_2O
fraction volume	10 ml

The sample used for gel filtration chromatography was filtered by the milipore filter of 0.45 μ and then condensed to 20 times by a rotary evaporator. Concentrated organics were separated by gel filtration chromatography after the condensed solution was filtered by 0.45μ filter. The loss of soluble organics by condensation was about 50 percent, and the recovery rate of soluble organics by gel filtration chromatography was over 90 percent.

From the gel filtration chromatography, the soluble organics are divided into four groups: Group I is composed of organics of over 1,500 molecular weight, Group II is consisted of organics with 260 nm adsorbance, Group III was organized by organics with no 260 nm adsorbance, and Group IV was made up of organics colored brown with 260 nm adsorbance.

When studying the gel filtration chromatography, the secondary effluent of the activated sludge treatment is generally low in TOC when compared with the primary effluent, and shows that activated sludge process is effective in removing the organics in Group III. When secondary effluent underwent a chemical precipitation process, organics in Group I were removed and those in Group II and Group III remained. However, when carbon adsorption is conducted, only a negligible amount of organics of Group IV was detected.

Table 1 shows the studies made on the chloroform formation in each group dosing chlorine to the samples divided into four at the gel filtration chromatography. The amount of chlorine dosed was about 10 mg Cl/ℓ. The reaction time was about one hour, and sodium sulfite was dosed to stop reaction. From Table 1, the following facts become clear:

Table-1 Chloroform Formation by Chlorination

	Not Chlorinated (μg/l)	Chlorinated (μg/l)	GFC Fraction (%)				
			Group I	Group II	Group III	Group IV	Total
Primary effluent	7.8 (40.6)	61.4	16.7	25.5	13.5	44.4	100
Secondary effluent	3.7 (14.8)	48.6	10.5	14.2	11.0	43.5	79.2
Coagulated* secondary effluent	1.8 (13.4)	39.5	4.9	16.2	10.3	32.9	64.3
Coagulated and adsorbed secondary effluent**	0.8 (7.7)	4.4	2.1	2.1	0	3.0	7.2

( ) Soluble TOC, mg/l

* Coagulant Alum, mole ratio Al/p = 3

** Coagulant Alum, mole ratio Al/p = 3
activated carbon Colgon Firtrasorb 300, SV = 5

1) When comparing the primary effluent and the secondary effluent, despite the TOC removal being 67 percent, the chloroform concentration only dropped by about 20 percent. Therefore, the removal of precursors can not be expected in the activated sludge treatment.
2) When comparing the secondary effluent and the coagulated secondary effluent, the chloroform concentration has dropped by 18.7 percent. Then, the coagulation process can not be considered that effective in removing precursors.
3) Carbon adsorption was most effective in the removal of precursors.
4) Group IV was brown in color and was considered to be formed by compounds resembling humic acid or fulvic acid. Of the chloroform formed in the primary effluent, the secondary effluent and the coagulated secondary effluent,

organics belonging to Group IV occupied nearly half.

5) Group II and Group IV organics with UV 260 nm adsorption produced a significant amount of chloroform. In the primary effluent, the chloroform produced by organics in Group II and Group IV accounted for nearly 70 percent of the total. Similarly, in the secondary effluent, they accounted for about 73% and in the coagulated secondary effluent about 76%.

4.1.2 Pilot Plant Tests

As shown in Table-1, activated carbon is quite effective in removing precursors.

As the pilot plant in Kyoto, studies on carbon capacity were made by feeding the effluent of the breakpoint chlorination process on the filtered secondary effluent and the filtered-adsorbed secondary effluent. As the test periods and the operation conditions of the carbon columns for both effluents differed, accurate comparison was not possible. The following shows the results of the tests:

Chloroform Concentration after Breakpoint Chlorination (µg/ℓ)

	average	range
filtered	36.2	11.9 ~ 80.5
filtered-carbon adsorbed	29.7	9.3 ~ 63.8

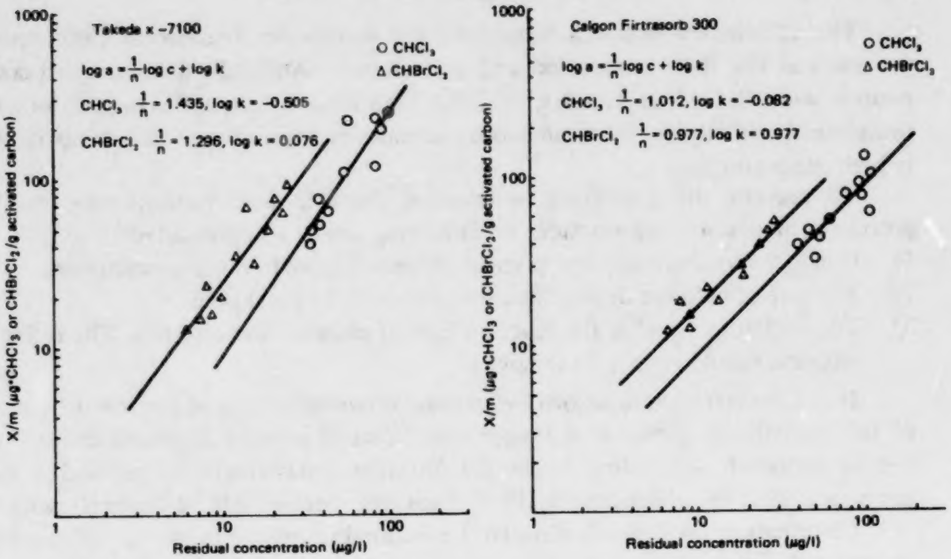
Studies of the results reveal that the residual chloroform concentration only dropped by 18 percent in the filtered-adsorbed secondary effluent compared with the filtered secondary effluent. At the pilot plant, the contact time of the carbon column was 36 minutes in a six-month continuous operation, and reactivation was conducted on once per six month basis. Accordingly, in this condition, the precursors in the wastewater within six month period seem to have broken through.

4.2 REMOVAL OF CHLORINATED ORGANICS

Fig. 4 shows the Freundlich's adsorption formula employing activated carbon of granulated Takeda X-7100 and Calgon Firtrasorb 300. The initial concentrations of chloroform and bromodichloromethane were set at 119 and 32 µg/ℓ, respectively. The temperature was between 24 and 25° centigrade.

From the Fig. 4, the following became clear:

Fig.-4 Freundlich's Adsorption Formula



1) Bromodichloromethane is adsorbed more easily than chloroform.
2) The adsorption capacity differed a little according to the types of activated carbon. But the chloroform per 1 gram of activated carbon or the adsorption capacity of bromodichloromethane (x/m) at the most was 100μg order.

At the Kyoto pilot plant, to remove the chloroform formed by the breakpoint chlorination process, tests were conducted by employing Takeda X-7000. The apparent contact time was 20.7 minutes. From the first week to the fourth week, the average removal rate of chloroform for each week was 80.0, 55.9, 53.9 and 40.4%, respectively. The chloroform adsorbed in the activated carbon during the operation in the 4th week was 54.8μg·$CHCl_3$/g. Accordingly, the chloroform adsorbed by the activated carbon was very small, thus removal of chloroform by activated carbon can not be highly expected.

The detected dichlorobenzene and trichlorobenzene in the effluents at the Kyoto plant were as follows:

Concentration (μg/ℓ)

	m·p-DCB	O-DCB	1.2.4.-TCB	1.2.3-TCB
Chlorination	2.2	3.2	0.48	0.45
Chlorinated-adsorbed	0.11	0.51	0.70	0.02

The concentrations of dichlorobenzene and trichlorobenzene were both μg/ℓ order. And except for 1.2.4-TCB, it was considered that the chloroform was comparatively easily adsorbed into the activated carbon.

5. SUMMARY

The chlorinated organics formed by the wastewater breakpoint chlorination process was low in concentration and polysucrose. Although the identified compounds were limited in number, of those identified trihalomethane (chloroform, bromodichloromethane, dibromochloromethane and bromoform) had comparatively high concentration.

To prevent the formation of trihalomethane in high concentration in the process of breakpoint chlorination, the following can be recommended:

1) Conduct carbon adsorption prior to chlorine dosing to remove precursors.
2) The rate of chlorine dosing should not exceed the breakpoint.
3) To shorten as possible the reaction time of chlorine and organics. The residual chlorine should readily be removed.

It is most effective to adsorb by granular activated carbon to remove precursors of trihalomethane. However, a long lasting effect of granular activated carbon can not be expected. According to the gel filtration chromatography method, it was anticipated that the organics with UV 260 nm adsorbance made up the precursors.

Chlorination on à disinfection level presented no special problems with regards chlorinated organics although there was possibility of bromine compound formation when the bromide ion concentration was high in the wastewater.

CHAPTER 3
DEWATERING AND INCINERATION OF SEWAGE SLUDGE WITH PULVERIZED COAL

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The dewatering and fluidized incineration method with pulverized coal is an object of public attention as a sludge incineration system that makes use of pulverized coal as dewatering aids, as substituted energy for oil and for prevention of secondary pollution.

Experiments of this dewatering and fluidized incineration had been carried out by Yokohama City* on a pilot plant, and fundamental data have been obtained. The outline of the experiments and the results obtained are reported below.

1. CHARACTERISTICS OF THE DEWATERING AND FLUIDIZED INCINERATION METHOD

This method is composed of the dewatering process and the incineration process, and is said to provide the following features.

In the dewatering process, dewatering is accomplished by making use of polymetric coagulant and pulverized coal to the level of moisture content that is identical to that of dewatered sludge obtained by using conventional inorganic coagulant. As a result, autogeneous combustion can be made in the incineration process without auxiliary fuel.

In the incinerating process, the fluidized bed is partially turned into a reductive atmosphere by red-hot pulverized coal, causing nitrogen oxide (NOx) to be reduced to nitrogen gas (N_2). At the same time, oxidization of chromium in the sludge into sexivalent chrome is prevented. In addition, by the use of cement clinker particles (major constituent CaO) as the fluidizing medium instead of silica which has been conventionally used, sulfur oxides (SOx) are retained as calcium compounds to be removed.

The major operation of this method is composed of operation of the dewaterer for obtaining autogeneously combustible dewatered sludge cake (control of dosage of coagulant and pulverized coal), control of the sludge loading rate (dewatered cake feed rate) and control of the air feed rate.

2. METHOD OF EXPERIMENTS

2.1 FLOW SHEET

A flow sheet of the pilot plant is shown in Figure 1, and specification of the principle equipment is shown in Table 1.

Thickened sludge (mixture of primary settled sludge and excess activated sludge) is stored in a storage tank, and is then fed to a mixing tank by a pump. Soluted polymetric coagulant is added to the thickened sludge and the mixing operation is carried out in a mixing tank for about 30 seconds to cause formation of flocs. The sludge is then thickened to two times by a rotary screen, and then fed to the next mixing tank, where the thickened sludge is mixed with pulverized coal in the form of slurry for about 30 minutes. The blended sludge is dewatered by a dewaterer of pressure filter type. The dewatered cake is fed to a hopper feeder for storage.

The dewatered cake is charged into the furnace from the top of the furnace at the standard feed rate of 510 $kg/m^2 \cdot h$ through a flight conveyor and a cake feeder. The temperature of the fluidized bed in the furnace is kept at the level of about 800°C, and the furnace outlet gas temperature is kept at about 900°C. The sludge makes autogeneous combustion. The air that is necessary for combustion is fed

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* The City made a contract on the experiment with Babcock Hitachi K.K., and set up a joint committee to carry out the experiment.

Fig. 1 Flow Sheet

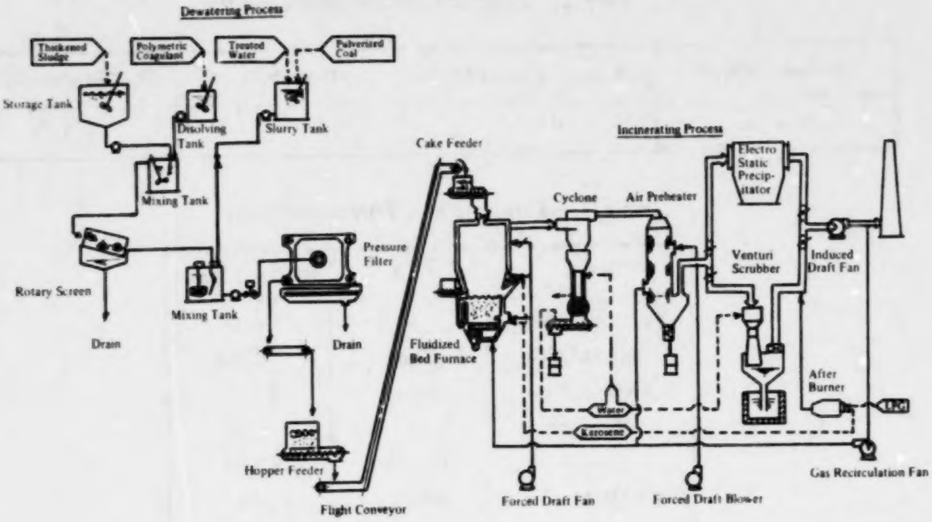


Table 1. Specification of Major Equipment

Process	Major Equipment	Specification
Dewatering Process	Pre-Thickener	Rotary Screen 900 mmφ x 1,770 mmL 0.75kW
	Sludge Feed Pump	Diaphram Pump 80 ℓ/min x 40m 5.5 kW
	Dewaterer	Pressure Filter 1,000 x 1,000 mm x 10 Chambers 16.7 m^2
Incinerating Process	Incinerator	Fluidized Bed Furnace 400 kg/h 1,000φ x 5,400 mmH
	Single Cyclone	High Temp. Type 2,400 Nm^3/h x 800°C
	Air Preheater	Tubular Heat Exchanger 16.7 m^2
	Wet Scrubber	Venturi Scrubber 2,400 Nm^3/h
	Electro Static Precipitator	Dry Type ESP 1,320 Nm^3/h
	Forced Draft Blower	Turbo Blower 40 m^3/min x 2,000 mmAq 30 kW
	Forced Draft Fan	Turbo Fan 90 m^3/min x 500 mmAq 15 kW
	Induced Draft Fan	Turbo Fan 190 m^3/min x 600 mmAq 37kW

through the air distributor located at the bottom of the fluidized bed. Recirculation of flue gas is made as required for controlling O_2 concentration in the furnace.

The majority of the dust in the flue gas is caught by a single cyclone. The flue gas enters an air preheater, after passing through the single cyclone, where the gas temperature is quenched to about 300°C, and is then fed to a venturi scrubber for wet scrubbing or to an electrostatic precipitator for electrostatic dedusting. The properties of polymetric coagulant, pulverized coal, and cement clinker particles used for the experiments are shown in Table 2, Table 3 and Table 4 respectively. Selection of these materials and the method of adding them were determined in laboratory experiments in advance.

2.2 EXPERIMENTS ON DEWATERING

The sludge used in the experiments was thickened sludge of two kinds of different VTS (volatile total solid), and the properties are shown in Table 5.

Table 2. Properties of Polymetric Coagulant

Molecular Weight	Viscosity of Solution (0.5%)	pH Available	pH of Solution (0.5%)
700 x 10^4	350 C.P.	<9	3.3

Table 3. Properties of Pulverized Coal

Items	Unit	Analyzed Value
Specific Gravity	–	1.39
Bulk Density	g/cm^3	0.758
Angle of Rest	degree	35
Moisture	%	5.38
Ash	% (D.B)	17.64
Gross Heating Value	Kcal/kg	6,210
C	% (D.B)	60.54
H	% (D.B)	4.66
N	% (D.B)	0.95
S	% (D.B)	0.48
$C\ell$	% (D.B)	0.01

Table 4. Properties of Fluidizing Medium

Specific Gravity (–)	Bulk Density (g/cm^3)	Angle of Rest (degree)	CaO (wt. %)	SiO_2 (wt. %)	$A\ell_2O_3$ (wt. %)
3.20	1.37	28	48.7	12.14	5.50

Table 5. Properties of Thickened Sludge

Kinds of Sludge	pH	TS (%)	Gross Heating Value (Kcal/kg)	Constituent of Solid (%)		
				C	H	N
High VTS Sludge (56%)	7.6	3.4	3,300	36.1	4.0	3.7
Low VTS Sludge (44%)	6.8	4.3	2,200	20.7	3.5	1.7

A small pressure filter in the bench scale (178 mm x 178 mm x 1 chamber) and a pressure filter in the pilot plant (1,000 mm x 1,000 mm x 10 chambers) were used for the experiments. In the experiments with a small pressure filter, dewatering was attempted with various combinations of coagulant and pulverized coal to investigate the effect of coagulant and pulverized coal in dewatering. With the pressure filter in the pilot plant, dewatering was made under the conditions identical to those with the small pressure filter, and investigation was made as for the scale-up effect of dewaterers. Dewatering conditions were filtration for 8 minutes at 4 kg/cm^2 and squeezing for 12 minutes at 10 kg/cm^2 for both pressure filters.

2.3 EXPERIMENTS ON INCINERATION

The heating value required for autogeneous combustion of sludge containing pulverized coal, the correlation between sludge loading rate, air feed rate and NOx concentration and the correlation between the supply rate of cement clinker particles and SOx concentration were investigated through the experiments. In addition, investigations were also made as for properties of ash, release of heavy metals and dust catching efficiency of electrostatic precipitator and of venturi scrubber.

3. RESULTS OF EXPERIMENTS

3.1 EXPERIMENTS ON DEWATERING

3.1.1 EXPERIMENTS ON THE SMALL PRESSURE FILTER

The results of experiments made with a small pressure filter are shown in Figure 2 and Figure 3. These figures represent moisture content of dewatered cake to the pulverized coal dosage (to total solid) for every coagulant dosage (to total solid). These figures also indicate the range in which removability of dewatered cake from filter cloth is good. In addition, these figures indicate the range in which dewatered cake autogeneously combusts, which will be described later.

According to the results of these experiments it is apparent that differences arise in the dewatering effect by the VTS of sludge.

a. Moisture content of dewatered cake

- For obtaining dewatered cake of identical moisture content, the dosage of polymetric coagulant and pulverized coal required for the sludge of low VTS is less than that required for the sludge of high VTS.
- In the case where VTS of sludge is high, the effect of the dosage of pulverized coal over the moisture content of the dewatered cake is not apparent if the pulverized coal dosage is 100% or less. In this case, however, a large difference occurs in the moisture content of dewatered cake when the difference in the dosage of polymetric coagulant is between 0.6% or less and 0.9% or higher.

Fig. 2 Results of Dewatering Experiment (VTS=56%)

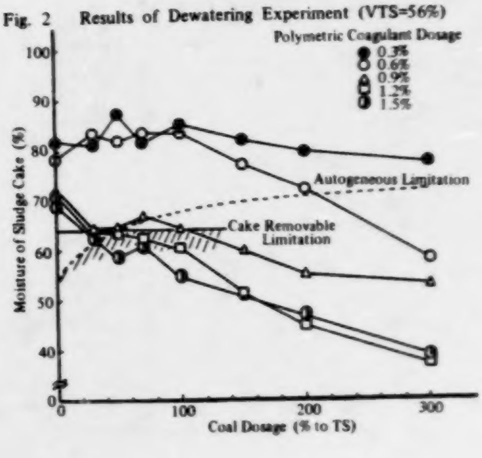
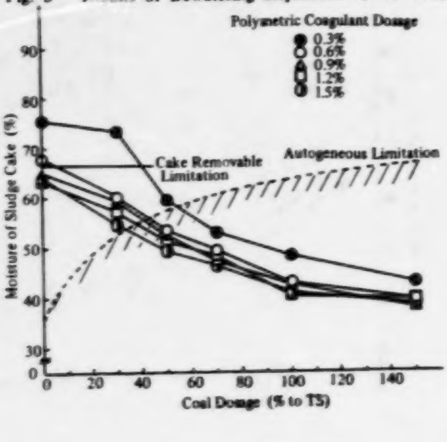


Fig. 3 Results of Dewatering Experiment (VTS=44%)



Where the dosage of pulverized coal is over 100%, the moisture content of the dewatered cake is reduced when the dosage of pulverized coal increases, even if the dosage of polymetric coagulant is fixed. In this case, reduction in the moisture content of dewatered cake due to the difference in the dosage of each coagulant is apparent to the dosage rate of 1.2%.

- In the case where VTS of sludge is low, the effect of pulverized coal over the moisture content of dewatered cake was observed from the range where dosage is small. As for polymetric coagulant, a difference in the effect over the moisture content of dewatered cake was observed between dosage rates of 0.3% and 0.6%, but no difference was observed in the moisture content of dewatered cake between other dosages.
- What indicates the moisture content of dewatered cake obtained through calculation by the simple addition of pulverized coal together with measured values is shown in Figure 4 and Figure 5. If the measured value is less than the calculated value which is indicated by broken lines, it is meant that pulverized coal caused reduction in moisture content of dewatered cake at a rate that is higher than the increase of solid materials, and it indicates that it has an effect as a dewatering aid.

  According to these figures it is found that measured values are less than calculated values and pulverized coal has an effect as a dewatering aid where VTS of sludge is low.

b. Cake removability

- As shown in Figure 2 and Figure 3, the removability of dewatered cake from the filter cloth is good below moisture content of cake of about 65% in the experiments.
- In the case where VTS of sludge is high, the dosages of pulverized coal required to obtain dewatered cake with removability are 300%, 150% and 30% at minimum respectively in correspondence to dosage of polymetric coagulant of 0.6%, 0.9% and 1.2 ∿ 1.5%.
- In the case where VTS of sludge is low, cake removability is good except for the cake where dosage rates of polymetric coagulant and pulverized coal are 0.3% and 30% respectively.

Fig. 4 Effect of Pulverized Coal as Dewatering Aids (VTS=56%)

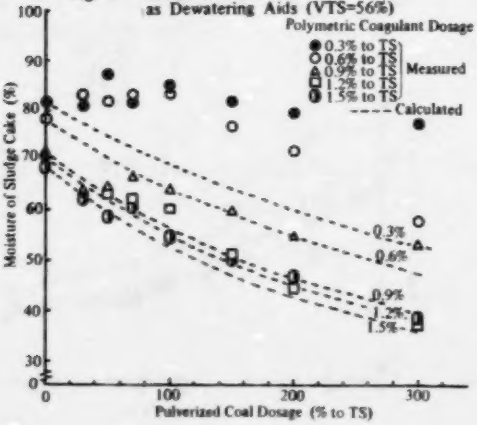
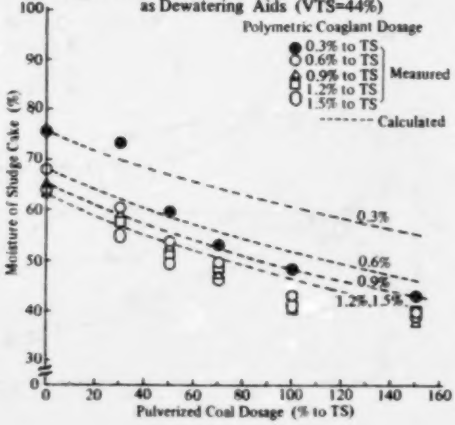


Fig. 5 Effect of Pulverized Coal as Dewatering Aids (VTS=44%)



3.1.2 EXPERIMENTS ON THE PILOT PLANT

a. Moisture content of dewatered cake

The results of experiments on the pilot plant which were carried out under the conditions identical to those of the fundamental experiments are shown in Figure 6. The moisture content of dewatered cake in the experiments on the pilot plant was higher in the range of 0 ∿ 4% than the results of the fundamental experiments, but the effect of polymetric coagulant and pulverized coal over dewatering was identical in both experiments.

b. Filtration rate

What indicates the filtration rate to VTS of sludge is shown in Figure 7. According to this figure, the filtration rate tends to be reduced as VTS of sludge increases.

3.2 EXPERIMENTS ON INCINERATION

3.2.1 CRITICAL LIMIT OF AUTOGENEOUS COMBUSTION

a. Limitation of heating value

The results of incineration of cake containing pulverized coal carried out with dosage of pulverized coal varied for the purpose of obtaining the heating value required for autogeneous combustion of the cake containing pulverized coal are shown in Table 6. The incineration temperature was 800°C and the excess air ratio was 1.3 at standard.

From Table 6, it was estimated that the critical heating value required for autogeneous combustion of dewatered cake is 1,100 Kcal/kg of wet cake because supplemental fuel combustion is required if the heating value is less than this level.

b. Limitation of moisture content and pulverized coal dosage

The autogeneous limits in said Figure 2 and Figure 3 give the autogeneous limit moisture content of cake containing pulverized coal to a certain pulverized coal dosage. The autogeneous limit moisture content was calculated based on the heating values of dry solid of sludge and of pulverized coal as well as pulverized coal dosage, assuming that the heating value required for

Fig. 6 Comparison of Sludge Cake Moisture between Bench Scale Pressure Filter and Pilot Plant Pressure Filter

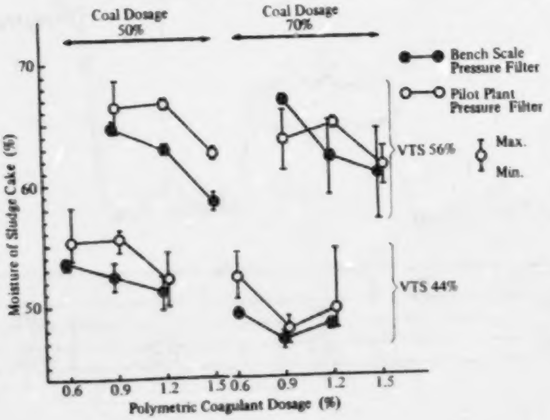


Fig. 7 Relationship between VTS of Sludge and Filtration Rate

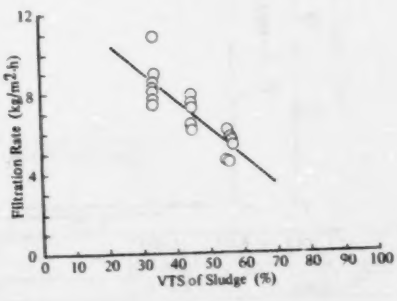


Table 6. Relationship between Heating Value and Autogeneous Combustion

Coal Dosage (% to Sludge TS)	Moisture of Cake (%)	Net Heating Value (Kcal/kg)	Kerosene Consumption (ℓ/h)	Bed Temperature (°C)
33.7	56.4	756	14.2	760
47.3	55.0	908	10.3	790
51.9	53.6	1,068	0	780
50.9	55.1	1,134	0	820

note) Thickened Sludge has 34 percents of VTS, 7.1 percents of TS and nearly 1,600 Kcal/kg of dry solid as heating value.

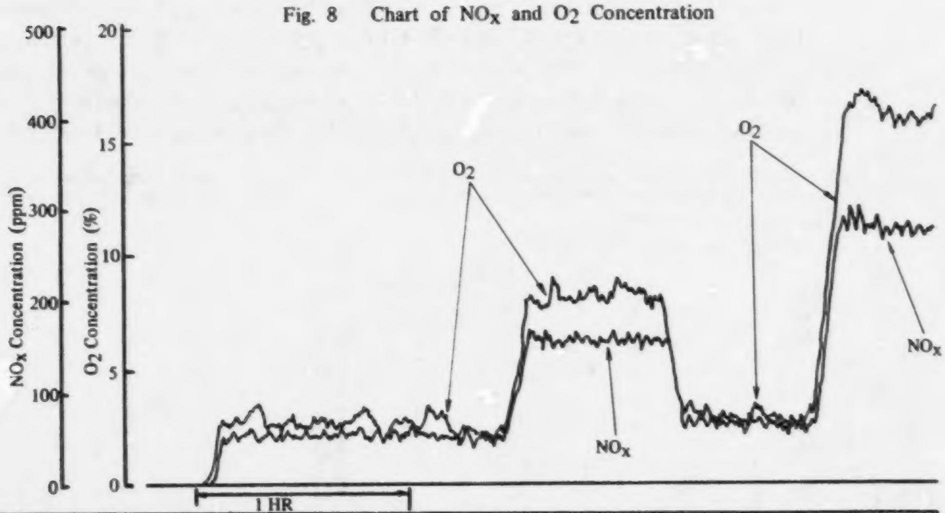
autogeneous combustion of cake containing pulverized coal is 1,100 Kcal/kg of wet cake.

From these figures, it is determined that dosages of pulverized coal of 50% or higher and dosages of polymetric coagulant of 0.9% or higher are required for sludge of high VTS (VTS 56%), and that dosages of pulverized coal of 50% or higher and dosages of polymetric coagulant of 0.3% or higher are required for sludge of low VTS (VTS 44%). Autogeneous combustion of dewatered cake cannot be accomplished if dosage of pulverized coal is 30% or less in this case. Therefore, from both aspects of removability of cake from filter cloth and autogeneous combustion, minimum dosage of pulverized coal is 50%, and the same of polymetric coagulant is 0.9% when VTS of sludge is high and 0.3% where VTS of sludge is low.

3.2.2 NOx CONCENTRATION

a. Sludge loading rate, air feed rate and NOx concentration

Fig. 8 Chart of NO_x and O_2 Concentration



Sludge Cake Feed Rate (kg/h)	400	300	300	200
Sludge Cake Loading Rate(kg/m^2h)	509	382	382	255
Fluidizing Air ($Nm^3/m^2 \cdot h$)	920	920	690	940

The change in the NOx concentration at the furnace outlet when the sludge loading rate and air feed rate are changed while continuous incineration of sludge is maintained is shown in Figure 8.

At the beginning of the experiments, the sludge loading rate was 509 $kg/m^2 \cdot h$ and an air feed rate was 920 $Nm^3/m^2 \cdot h$, and then the sludge loading rate was reduced by 25% to 382 $kg/m^2 \cdot h$ with the air feed rate unchanged. As a result, O_2 concentration at the furnace outlet increased and NOx concentration also increased accordingly. Next, it was attempted to reduce the air feed rate by 25% to 690 $Nm^3/m^2 \cdot h$ with the sludge loading rate kept unchanged. As a result, O_2 concentration at the furnace outlet was reduced and NOx concentration was also reduced. Then, the air feed rate was increased to the original value and the sludge loading rate was reduced to 50% of the original value. A remarkable increment in the O_2 concentration and the NOx concentration occurred as a result.

From these results, it can be said that the NOx concentration is related to both the sludge loading rate and the air feed rate and that it is possible to reduce NOx concentration as the sludge loading rate is increased to the air feed rate, that is, as O_2 concentration at the furnace outlet is reduced.

b. Relationship between O_2 concentration and NOx concentration at the furnace outlet

The relationship between O_2 concentration and NOx concentration at the furnace outlet is shown in Figure 9. There is a tendency that the lower the O_2 concentration, the lower the NOx concentration. The relationship between the air feed rate/sludge loading rate and O_2 concentration is shown in Figure 10. O_2 concentration is reduced as the air feed rate/sludge loading rate is low. It appears to be possible to control the NOx concentration by regulating the sludge loading rate or the air feed rate.

Fig. 9 NO_x Concentration vs. O_2 Concentration

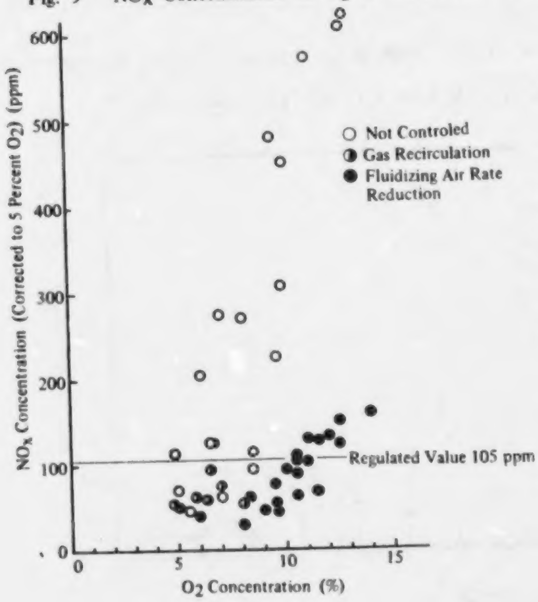
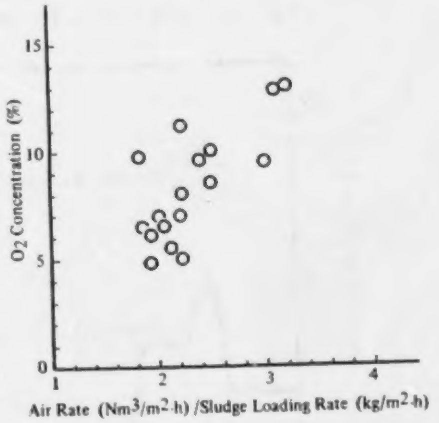


Fig. 10 O_2 Concentration vs. Air Rate/Sludge Loading Rate



c. Flue gas recirculation and NOx concentration

Experiments were made to control O_2 concentration by causing a part of flue gas released to the atmosphere through a chimney to be recirculated into cumbustion air and by mixing it with fluidizing air. The relationship with O_2 concentration in the case where flue gas recirculation is made is shown in said Figure 9. When flue gas recirculation is made, NOx concentration is reduced even if O_2 concentration remains unchanged. On the other hand, however, there is a tendency that CO concentration in the flue gas increases, and that uncombusted matters in the ash increase.

d. Reduction of fluidizing air rate and NOx concentration

For the NOx reduction, the fluidizing air is lowered to 0.9 ∿ 1.2 of a theoretical amount of air so that the fluidized bed may become more reductive atmosphere. In this case, unburnt materials are combusted by supplement of fresh air supply to the freeboard of the furnace.

As the results shown in Figure 9, when the fulidizing air rate is reduced, NOx concentration is decreased even if O_2 concentration remains on a same level. Reduction of the fluidizing air rate is effective in NOx removal as well as flue gas recirculation.

3.2.3 SOx CONCENTRATION

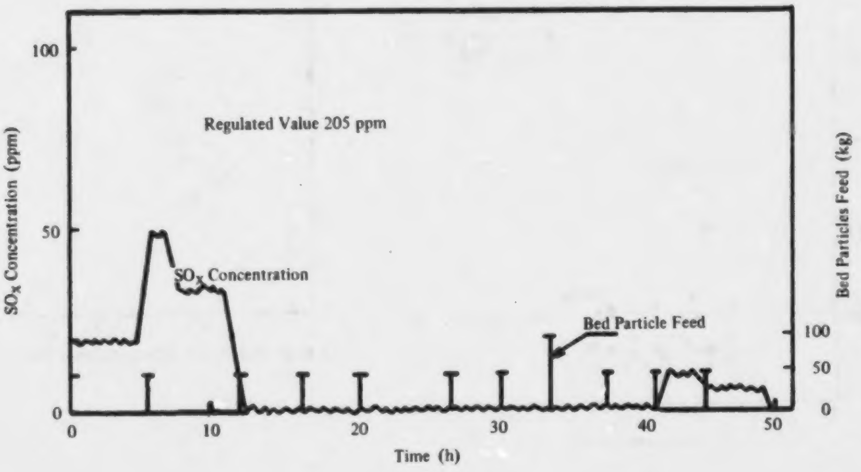
Figure 11 illustrates the change of SOx concentration in the furnace when the furnace is continuously operated for 50 hours while supplying cement clinker particles by a fixed rate (0.025 kg per 1 kg of sludge cake). SOx concentration was as high as 50 ppm when cement clinker particles were initially supplied, but it never exceeded 10 ppm thereafter.

3.2.4 HEAVY METAL RELEASE FROM ASH

The particle size of ash was 0.5 mm at maximum and 0.1 mm at average. Ignition loss was about 1% in an ordinary case and was 3 ∿ 4% when flue gas recirculation was made.

According to the results of heavy metal release experiments, even if Cr^{6+} was

Fig. 11 Relationship between Bed Particle Feed Rate and SO_x Concentration



detected partly at 0.19 mg/ℓ, the majority was of the limit of determination (0.02 mg/ℓ) or less. Other heavy metals, i.e., Fe, Mn, Cu, Zn, and Pb were not detected. Concentration of Hg and As, which are heavy metals of low boiling point is shown in Table 7. It is noted that these metals are vaporized and dispersed in flue gas, although the rate is minor.

The ash obtained through these experiments contains Ca by 5 ∿ 15%. The extract obtained in the extraction test indicates the pH of 11 ∿ 12, and it is considered that heavy metals are in hardly releasable condition. The results of a test of releasing of Cr^{6+}, Cd and Pb with pH value varied for the purpose of warranting safety of ash at the disposal site are shown in Figure 12. According to these results, releasing of these metals does not occur even if the extract of ash is of weak acidity.

3.2.5 DUST CONCENTRATION

The results of investigation of dust catching efficiency with two kinds of dust catchers, that is, a dry electrostatic precipitator and a wet venturi scrubber are shown in Table 8 and Table 9. Dust catching efficiency is higher with the electrostatic precipitator, but dust concentration of both dust catchers satisfies the regulated value.

Table 7. Hg and As Concentration in Flue Gas

Location		Cyclone Outlet						EP Inlet	EP Outlet
Date		Mar. 30, 1977						Mar. 31	Mar. 31
Time		9:30	10:00	10:30	16:45	17:15	17:40	15:30	15:30
Hg	(mg/Nm^3)	0.18	3.05	2.67	2.16	1.48	0.17	0.15	0.19
As	(mg/Nm^3)	0.58	N.D.	0.20	0.22	0.19	0.70	N.D.	N.D.

Fig. 12 Relationship between pH of Extract and Metal Release

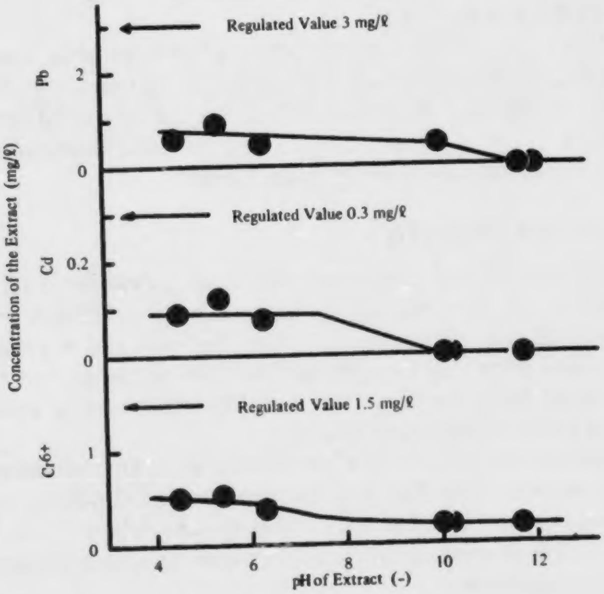


Table 8. Dust Collecting Efficiency of Venturi Scrubber

Date and Time	Location	Dust Concentration (g/Nm^3)	Collecting Efficiency (%)	Dust Concentration (Regulated Value)
1977 Mar. 29 18:15 ~ 20:10	Venturi Inlet	7.6		0.2 g/Nm^3
1977 Mar. 29 18:50 ~ 20:10	Outlet	0.063	99.17	
1977 Mar. 30 16:23 ~ 17:28	Venturi Inlet	6.5		
1977 Mar. 30 17:30 ~ 18:25	Outlet	0.021	99.67	

note) Dust concentrations shown are mean values found during the time of measurement.

Table 9. Dust Collecting Efficiency of Electro Static Precipitator

Date and Time	Location	Dust Concentration (g/Nm^3)	Collecting Efficiency (%)	Dust Concentration (Regulated Value)
1977 Apr. 4 16:15 ~ 17:00	EP Inlet	2.441		0.2 g/Nm^3
	Outlet	0.0059	99.75	
1977 Apr. 4 17:10 ~ 17:50	EP Inlet	4.520		
	Outlet	0.0097	99.78	
1977 Oct. 25 14:00 ~ 14:40	EP Inlet	5.716		
	Outlet	0.0002	99.99	
1977 Oct. 27 14:10 ~ 14:50	EP Inlet	7.912		
	Outlet	0.009	99.99	

note) Dust concentrations shown are mean values found during the time of measurement.

3.2.6 MATERIAL BALANCE

Material balance for the case of 100 tons of sludge solid based on the results obtained through these experiments is shown in Figure 13. According to Figure 13, the weight of ash after being humidified is 1/5.6 of that of dewatered cake and its weight is 1/6.0 of that of cake. These figures indicate that it is possible to reduce the weight of dewatered cake through incineration.

4. SUMMARY OF RESULTS

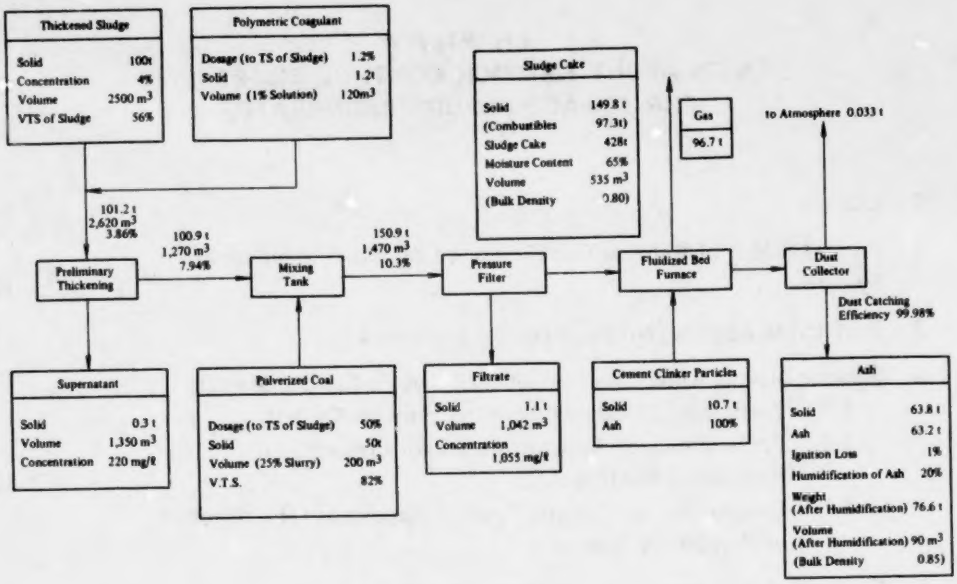
Thickened sludge was dewatered by using pulverized coal and polymetric coagulant, and it was possible to obtain dewatered cake that combusts without supplementary fuel. The dosage rates of pulverized coal and of polymetric coagulant which are matched with this method vary by VTS of sludge, but they are 50% and 0.3 ∿ 0.9% respectively from the removability of dewatered cake and also from critical limit of autogeneous combustion.

NOx concentration in the flue gas is reduced as O_2 concentration at the furnace outlet is reduced, and flue gas recirculation and fluidizing air rate reduction were also found effective in reduction of NOx concentration.

It was possible to control SOx concentration by supplying cement clinker particles as fluidizing medium.

As for heavy metal release from ash, even if Cr^{6+} was partly detected, other metals were not detected.

Fig. 13 Material Balance



CHAPTER 4
FACTS ABOUT EMISSION CONTROL EQUIPMENTS FOR SEWAGE SLUDGE INCINERATOR

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1. OUTLINE

The recent two years survey has shown that the treatment of restricted hazardous materials in the exhaust gas discharged by incineration of sewage sludge is being dealt with effectively, meeting all the existing standards for restriction of such materials. For collection and removal of particulate matters and sulfur oxide (SOx), the used of scrubbers was found to be particularly effective.

It has also been learned that these hazardous materials can be more effectively eliminated by using an electrostatic precipitator and an "alkaline liquid absorption system", in conjunction with the scrubbers. This will make it possible to use the system in areas where more stringent exhaust gas standards are applied.

No specific counter-measurers have yet been proposed to deal with the recent (June, 1977) restriction of nitrogen oxide (NOx). However, experiments for its effective elimination have been made and measures which we believe to be effective have been developed.

This paper is based upon the results of our investigation and reports on the following items:

(1) Performance of Apparatus for Collection of Particulate Matters
(2) Controlled Incineration at a Multiple Hearth Furnace (Capacity: 50 ton/day)
(3) Restriction of NOx at a Fluidized Bed Incinerator (Capacity: 40 ton/day)
(4) Reduction of NOx by means of Ammonia (NH_3) Injection at a Fluidized Bed Incinerator equipped with Pre-drying Heaters (Capacity: 36 ton/day)

2. PRESENT STATUS OF EXHAUST GAS GENERATED BY EXISTING INCINERATION SYSTEMS

Table 1 shows the results of analysis made of the exhaust gas discharged by incineration of sewage sludge at a conventional multiple hearth furnace and a fluidized bed incinerator. The measurements were taken at the furnace outlets and then statistically compiled. Data includes actual test data by our Testing Division as well as those supplied by furnace manufacturers in response to our questionaires. Three kinds of dewatered cakes were fed to the furnaces; those dewatered with vacuum filters, filter presses and centrifuges. However, due to scarcity of data for incineration of centrifuge-dewatered cakes at multiple hearth furnaces, they were excluded from the Table.

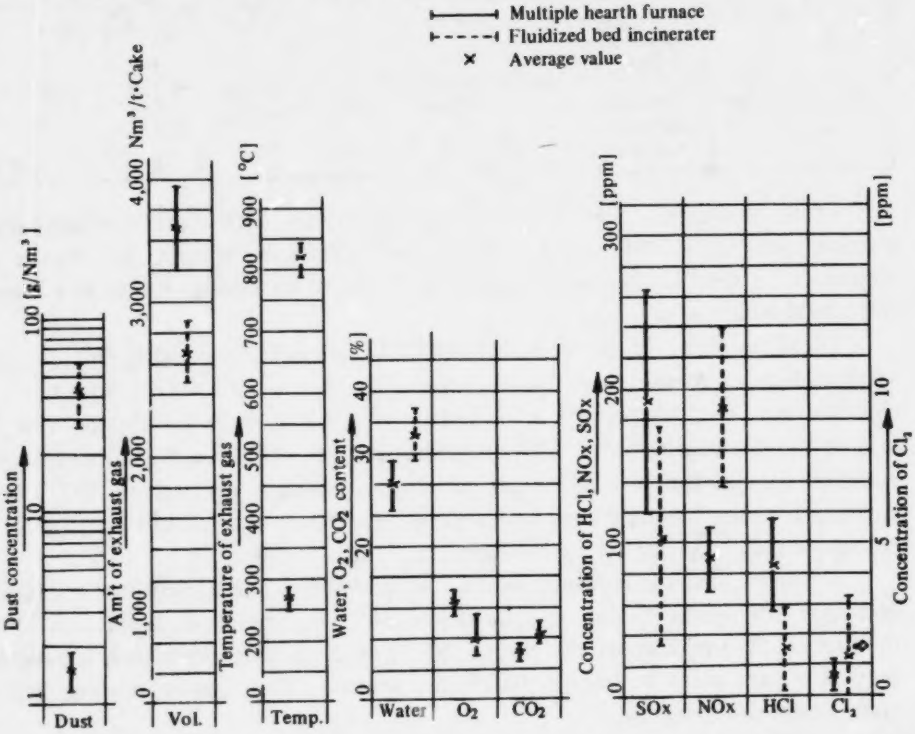
Table-1 Most Probable Properties Exhaust Gas Broken Down by the Type of Furnace

		Multiple Hearth Furnace						Fluidized Bed Incinerater							
		Vacuum Filter		Filter Press		Total		Vacuum Filter		Filter Press				Total	
		Average Value	Standard Deviation	Average Value	Standard Deviation	Average Value	Range	Average Value	Standard Deviation	Average Value	Standard Deviation	Average Value	Standard Deviation	Average Value	Range
Waste Gas Temp.	°C	270.7	47.6	235.0	33.8	266.7	247~286	823.8	55.0	807.1	68.5	812.5	62.9	816.5	791~842
Am't of Waste Gas	Nm^3/t Cake	3,673	641	2,778	431	3,491	3,211~3,771	2,510	265	2,726	512	2,982	587	2,672	2,473~2,871
Moisture Content	%	26.1	8.1	30.5	10.5	27.8	24.5~31.1	33.8	7.8	33.7	7.7	36.7	7.8	34.3	31.0~37.6
O_2	%	13.5	2.4	10.0	3.0	12.6	11.3~13.8	8.3	4.1	6.5	2.6	5.9	1.4	7.4	5.8~9.0
CO_2	%	5.3	1.9	8.6	2.3	6.2	5.1~7.3	7.5	2.3	8.7	1.3	12.9	1.3	8.6	7.1~10.0
Dust Concentration	g /Nm	1.46	1.00	1.81	0.72	1.57	1.20~1.94	38.3	24.4	79.1	19.1	15.0	6.2	44.5	27.6~61.4
SOx	ppm	126.3	161.2	313.4	201.2	191.2	118~264	49.6	74.5	99.3	108.3	325	263	103.4	32.7~174.1
HCl	ppm	80.8	80.6	95.3	44.3	85.0	55~115	26.2	47.5	30.6	47.9	–	–	30.3	3.4~57.2
Cl_2	ppm	0.80	1.33	0.41	0.64	0.66	0.12~1.2	0.39	0.74	3.3	5.8	–	–	1.15	0~3.2
NOx	ppm	100.6	39.2	48.6	37.9	88.7	68~109	188.0	73.1	261.6	73.1	54.7	37.7	186.4	135~238
Odor Concentration	–	5,597	8,346	2,241	2,852	3,912	•	–	–	–	–	–	–	–	–
No of Experiment		19 Case		8 Case				10 Case		7 Case		4 Case			

Fig 1 is the illustration of Table 1. It shows that the concentration of particulate matters emitted by a fludized bed incinerator is higher than that of the multiple hearth furnace, as was expected from the differences in their systems. Also, the cake dewatered by centrifuge using a polymar as filter aid shows lower concentration at the furnace outlets than cakes make with vacuum filters or filter presses using inorganic coagulant.

It was found that the emission of SOx and HCl can be greatly reduced if lime is used as filter aid, because most of the sulfur and chlorine are fixed in the lime. For example, the residual contents of sulfur and chlorine in the ash were between 77 to 89% and 78 to 88%, respectively, of their initial contents in the cakes. This can be readily understood from the fact that lime tends to fix sulfur, as evidenced from the use of magnesium as sulfur fixing agent in the Eschka's method which is used to measure sulfur contents in the fuel, and also from the fact that chlorine tends to fix lime.

Fig.-1 Properties of Waste Gas of Multiple Hearth Furnace, Fluidized Bed Incinerater



3. RESTRICTION OF NOx (NITROGEN OXIDE) EMISSIONS

The Third Air Pollution Prevention Act. enacted on June 16, 1977, for the first time set a limit on the emission of NOx generated by sludge incineration, and stipulated that installations which emit more than 40,000 Nm^3/h of exhaust gas must restrict the amount of NOx emission to be less than 250 ppm (Converted on the basis of $O_2 = 12\%$). For future installations, this goes into effect on June 18, 1977. Existing facilities which have to be renovated to meet the NOx emission criteria were given a three-year grace period, and the dead line will be May 1, 1980.

The amount of sludge incineration which corresponds to the above-mentioned 40,000 Nm^3/h of exhaust gas can be estimated as follows:

No.	Place	Kind of Sludge	Water Content % W.B	Combustibuls % W.B	Incineration t/h Corres. to 40.000 Nm^3/h Exhaust Gas
1	A Sludge	Centrifuge Dewatered Cakes	86.58	0.16	12.3
2	B Sludge	Filter Press Dewatered Cakes	60.76	0.15	15.1
3	C Sludge	Centrifuge Dewatered Cakes	78.56	0.15	11.8

From the table, it can be seen that installations with the design capacity of burning more than 250 tons/day come under the above restriction. This means that almost all of the facilities in large cities such as Tokyo, Osaka, Kyoto and Nagoya fall into this category.

There are, however, municipalities which have enacted more stringent standards of their own. Kawasaki City, for example, set a limit on the total amount of NOx emission that may be permitted for a given area, based upon calory values of fuel used or the sludge incinerated. This municipal code specifies that the amount of NOx emission per one ton of sludge must be less than 800 g, converted into NO_2 for the existing facilities having a capacity of more than 50 tons/day, and less than 0.19 g, converted into NO_2 per 0.15×10^{10} kcal for future facilities.

Assuming that the concentration of suspended solids in the influent is 250 mg/l and that the entire sludge is conditioned with inorganic coagulant, the above-mentioned 50 tons/day of the sludge cake would equal to one which is produced from a sewage plant to handle 40,000 m^3 sewage a day. Many existing facilities come under this category.

0.15×10^{10} kcal is even more strict, extending the coverage to facilities of much small scale, as illustrated by the following calculation:

Assuming that;
Fuel consumption per one ton of sludge 50l
Calorific consumption per one liter of fuel .... 8,000 kcal

Then, the calorific consumption per year for an installation with a capacity of combusting 50 tons of sludge/day will be as follows:

50 ton/day x 50 l x 8,000 kcal x 365 days/year = 0.73×10^{10} kcal/year

Thus, the 50 ton/day capacity consumes more calories than 0.15×10^{10} kcal/year.

As enumerated above, the restrictions on NOx emissions are becoming increasingly stringent.

4. PERFORMANCE OF APPARATUS FOR COLLECTION OF PARTICULATE MATTERS

4.1 PARTICULATE EMISSIONS FROM THE FURNACE OUTLETS

From the results of the surveys conducted to-date, the present status of the particulate emissions at the furnace outlets are as follows:

	Multiple Hearth Furnace	Fluidized Bed Incinerator
Range of Particulate sizes μ	1 ~ 140	5 ~ 140
Range of Medium sized Particles μ	2 ~ 10	10 ~ 60
True Specific Gravity	2.80 ~ 2.97	2.35 ~ 3.01
Apparent Specific Gravity	0.50	0.61 ~ 0.76

Fig. 2 and 3 show, in Rosin Rammler diagrams, the distribution of particle sizes, measured from the samples obtained at the outlets of multiple hearth furnaces and fludized bed incinerators, respectively.

In the case of fluidized bed incinerators (Fig. 3), some similarities in the particle distribution patterns can be observed from the straight slanting lines, although the particulate sizes vary. Generally speaking, however, the dispersion patterns vary in both types of furnaces, indicating no set patterns.

In the Fig. 3, YS denotes the particulate emissions generated by incinerating cakes which used polymer as a filter aid, whereas HK and GH dennote particulate emissions from cakes in which lime was used as a filter aid. As noted, YS, which used polymer, shows finer particle sizes.

In the case of multiple hearth furnace (Fig. 2), all the cakes incinerated used lime as a filter aid; none used polymar. Generally fine particulate sizes are shown. The reason why that, in the fluidized bed incinerator, the particle sizes tend to be coarse when lime was used as filter aid, is because the particulate sizes of the lime are coarser than those of the sludge itself, thus creating shift of particle sizes to those of the larger.

On the other hand, the movement of exhaust gas in the case of the multiple hearth furnace is slower than that of the fluidized bed incinerator. Also, since there is no crushing of sludge, as in the case of fluidized bed incinerators, only the fine particulate matters are emitted. These seem to account for emission of finer particulate sizes than in the case of fluidized bed incinerator. The typical example of the above is observed with TS of the multiple hearth furnace.

Fig.-2 Rosin-Rammlar Distribution Chart of Dust at the Outlet of Multiple Hearth Furnace

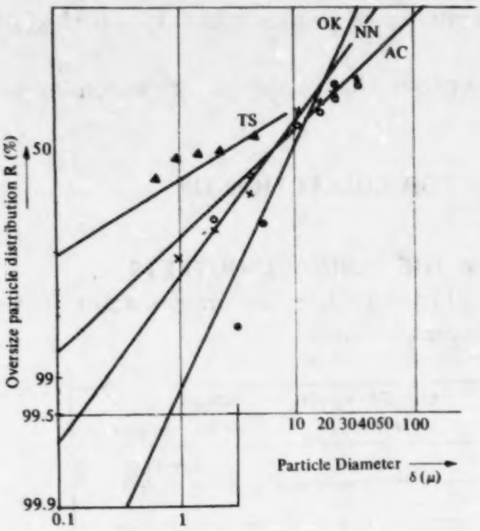
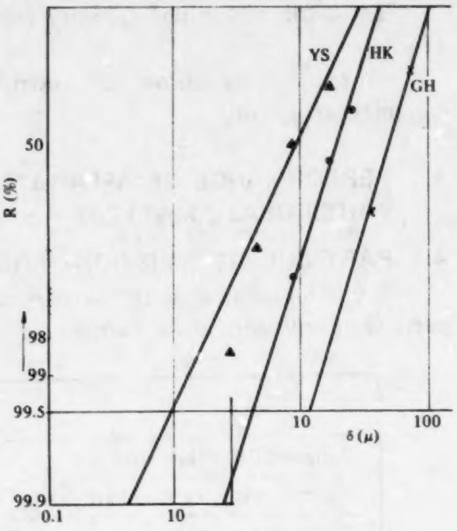


Fig.-3 Rosin-Rammlar Distribution Chart of Dust at the Outlet of Fluidized Bed Incinerator



4.2 PERFORMANCE OF APPARATUS USED FOR COLLECTION OF PARTICULATE MATTERS

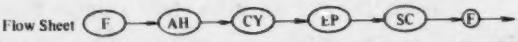
Tables 2, 3 and 4 show the performance of apparatus for collection of particulate matters used with multiple hearth furnaces and fluidized bed incinerators. The cakes fed to the multiple hearth furnace (TS) and fluidized bed incinerator (GH) used lime as a filter aid, whereas those fed to the fluidized bed incinerator (YS) used polymer.

Table-2 Multiple Hearths Furnace TS

Flow Sheet (F) → (FSC) → (AT) → (EP) → (F) →

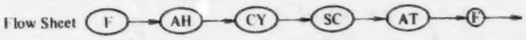
		Temp. of exhaust gas	Static pressure	Moisture	Flow volume of wet gas	Dust concentration	Dust collection efficiency	Remakrs
		°C	mm H_2O	%	Nm^3/hour	g/Nm^3	%	
Furnace outlet	1	250	−38	26.0	46,800	2.2	–	
	2	306	−57	33.1	46,000	2.2	–	
Fluidized bed scrubber (outlet)	1	22	−925	2.9	34,800	0.13	94	Am't of scrubbing water
	2	30	−571	4.4	39,000	0.22	88	
Fluidized bed absorption tower (outlet)	1	15	−1,156	1.9	31,000	0.095	–	
	2	20	−925	2.5	40,000	0.14	–	
E.P. outlet	1	28	+45	3.7	35,900	0.009	89	Wet type EP 17m^3/hour
	2	32	+82	4.7	42,000	0.021	84	

Table-3 Fluidized Bed Incinerator GH



		Temp. of exhaust gas	Static pressure	Moisture	Flow volume of wet gas	Dust concentration	Dust collection efficiency	Remarks
		°C	mm H_2O	%	Nm^3/hour	g/Nm^3	%	
Furnace outlet (after secondary heat exchange)	1	355	−140	39.0	5,300	78.9	–	
	2	362	−80	43.5	5,300	72.6	–	
Cyclon outlet	1	240	−200	37.6	5,400	6.2	91	
	2	265	−120	42.0	5,300	7.1	89	
EP outlet	1	150	−250	18.2	7,400	0.52	84	Dry-type EP
	2	176	−185	19.3	7,400	0.53	85	
Spray/scrubber outlet	1	35	−400	7.6	6,500	0.008	98	Volume of scrubbing water 58m^3/hour
	2	30	−270	4.6	5,300	0.017	97	

Table-4 Fluidized Bed Incinerator YS



		Temp. of exhaust gas	Static pressure	Moisture	Flow volume of wet gas	Dust concentration	Dust collection efficiency	Remarks
		°C	mm H_2O	%	Nm^3/hour	g/Nm^3	%	
Furnace outlet	1	525	−2	47.2	3,890	20	–	
	2	–	–	–	–	–	–	
Cyclon outlet	1	442	–	33.7	3,470	3.4	81	
	2	482	−10	35.3	3,490	9.7	–	
Venturi scrubber outlet	1	10	–	1.0	2,530	0.06	98	Volume of scrubbing water 85m^3/hour
	2	19	−36	4.5	1,940	0.084	99	

(AH) : Heat exchanger (CY) : Cyclon (Dry type) (CP) : Electrostatic precipitator (AT) : Absorption tower
(F) : Furnace (SC) : Scrubber (I) : Induction fan (FSC) : Fluidized bed scrubber

(1) Cyclon (Dry Type)

Since the particulate emissions from the fluidized bed incinerator normally have higher concentration and larger particle sizes than those from the multiple hearth furnaces, the use of a cyclon (dry type) which employs centrifugal force is effective for collection and removal of the particulate matters.

	Concentration of P.M. at Inlet g/Nm^3	True Specific Gravity	Pressure Drop mm-H_2O	Rate for Collection of P.M. %
Fluidized Bed Incinerator GH	73 ~ 79	3.01	40 ~ 60	89 ~ 91 (Lime)
Fluidized Bed Incinerator YS	20	2.35	110	81 (Polymar)

Also, as seen from the above table, GH which combusted cakes for which lime was used as a filter aid shows higher emission concentration but less pressure drop, as compared with YS which used polymer as a filter aid, and had a higher collection rate of particulate matters.

(2) Scrubbers

Scrubbers play an important role in the purification of exhaust gas generated by sludge combustion. They not only collect and remove particulate matters, but also prove to be effective for cooling and absorption of water-soluble gas.

There are several types of scrubbers. The packed tower type serubber is not normally used with sludge incineration as it tends to jam. A spray tower type as well as venturi type scrubbers are normally used with sludge incineration. The table below shows the performance of each type of scrubbers, based upon the data we have gathered.

	Concentration of P.M. at Inlet g/Nm^3	Pressure Drop mm-H_2O	Rate for Collection of P.M. %	Liquid-gas l/m^3
Spray Tower	0.52 ~ 0.53	85 ~ 150	84 ~ 85	8
Venturi Scrubber	3.4 ~ 9.7	350	98 ~ 99	24
Aerotic Floating Bed	2.2	500	88 ~ 94 (3 Beds)	5

The rate of collection of particulate matters of the spray tower type scrubbers is lower than that of the venturi type, though this may be attributable to the lower concentration of particulate matters at the inlet. However, the pressure drop is small and so is the liquid-gas ratio. The venturi type scrubbers, on the other hand, shows its suitability for use with particulate matters of high concentration, and the collection and removal rate is also high. However, its pressure drop is great and requires a greater quantity of scrubbing water.

The aerotic floating bed scrubbers is the type which fluidizes the packing and is highly effective for collection of fine particles, requiring only a small quantity of scrubbing water.

(3) Electrostatic Precipitator

An electrostatic precipitator collects particulate matters in the exhaust gas by charging the particles electrostatically and then separating them from the gas by an electrical force. There are two types for this system; wet and dry. The dry type collects the particles from the collecting electrodes by striking the electrodes intermittently with a hammer. The wet type collects the particles by a stream of water on the electrode surface.

The effectiveness of the electrostatic precipitation varies depending upon its inlet condition; viz., the conditions of gas or particulate matters. Hence, a study of the exhaust gas condition at the inlet is necessary in order to maintain high efficiency.

Amontg the factors which are present at the inlet, the one which affects the efficiency of collection of particulate matters and the operational stability is the electrical resistance of the particles. Generally speaking, the particles having electrical resistance within the range of $10^5 \sim 10^6 \Omega$ cm to $10^{10} \sim 10^{11} \Omega$ cm can be dealt with rather easily.

Experiments were made to compare the apparent electric resistance of particulate emissions from incinerating cake conditioned with inorganic coagulant (No. 1) and with polymer (No. 2). The results were as follows:

Voltage: 2,000 V/cm
Water Content: 0%
Temperature: 150 ~ 200°C

			Apparent Electric Resistance
KI (No. 1)	Particulate Matters	Inorganic Coagulant	$2.44 \times 10^{12} \sim 1.06 \times 10^{12}$ cm
KI (No. 2)	Particulate Matters	Polymar	$1.30 \times 10^{11} \sim 2.38 \times 10^{10}$ cm

Both of the cases showed apparent electric resistance higher than the desired $10^{10} \sim 10^{11}\,\Omega$ cm, though some differences were observed. Some measures to lower the apparent electric resistance are considered necessary.

Generally speaking, the apparent electric resistance tend to maximize in gas temperatures of 150 to 200°C, and decrease as the water content increases. With the dry type electrostatic precipitator, therefore, the adjustment of these two factors is considered necessary.

Fig. 4 shows dispersion of particulate emissions at inlets of KI (No. 1) and KI (No. 2). As clearly seen, the particle sizes are smaller with No. 1 (lime).

	Gas Flow Orientation	Press. Drop mm-H_2O	Water C. Exhausts Gas at Inlet %	Exhaust Gas Temp. °C	Concent. of P.M. g/Nm^3	Collect. Rate of P.M. %
Dry Type GH	Horizontal	50 ~ 65	38 ~ 42	240 ~ 270	6.0 ~ 7.0	84 ~ 85
Dry Type KI (No. 1)	Horizontal	10	16	250	1.6	95
Dry Type KI (No. 2)	Horizontal	10	25	300	0.3	86
Wet Type TS	Vertical	20 ~ 60	2 ~ 3	15 ~ 20	0.09 ~ 0.14	84 ~ 89

Of the dry types, KI (No. 1) shows the best collection rate of 95%. KI (No. 1) and KI (No. 2) are of identical design and the water content as well as the temperature of the exhaust gas at the inlet are considered to be approximately the same. The reason why KI (No. 2) shows a lower collection performance despite its high efficiency is presumably due to the lower concentration rate of particulate matters (0.3 g/Nm^3), as compared with the concentration rate of KI (No. 1) (1.6 g/Nm^3). Conversely, GH presumably registered a low collection performance, as the concentration was at a high level of 6 ~ 7 g/Nm^3.

For the wet type, only one example at the TS incinerator is shown. Despite the low concentration rate of 0.1 g/Nm^3 at the inlet, a rather high collection, average of 86%, was recorded. The wet type seems to have a better collecting performance, particularly for collection of extremely small particulate matters. This is due to a constant flow of water on the collecting electrodes, eliminating such disadvantages as (1) re-flying of the particulate matters that had once stuck to the electrodes, and (2) the failure of particles to separate from the electrodes, even with hammering, as is often the case with the dry type precipitators.

Fig. 5 shows the dispersion of particulate matters at the inlets and outlets of the GH (dry type) and the TS (wet type) electrostatic precipitators. It can be seen that the TS processed extremely fine particles.

Fig.-4 KI Incinerator Diagram of RRS Particle Sizes of Particulate Matters of EP Outlet

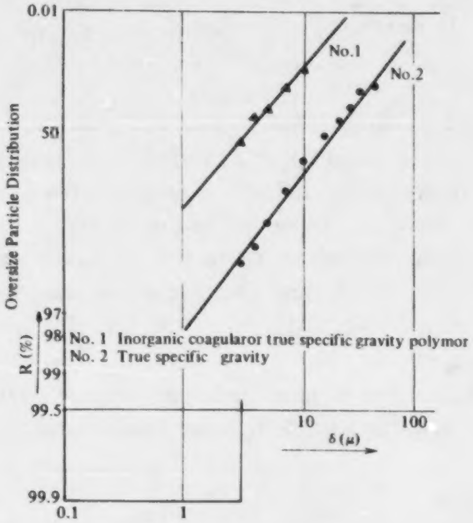
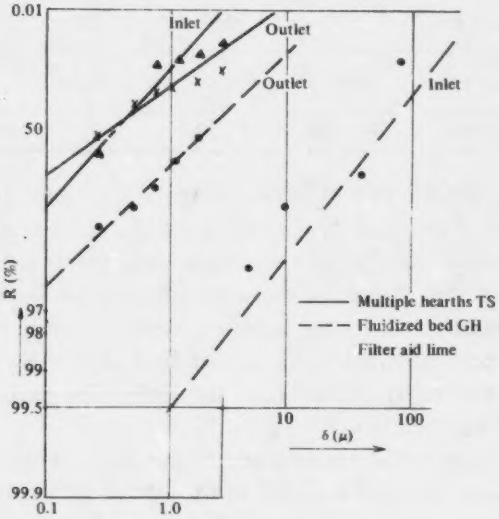


Fig.-5 Diagram of Particle Sizes of Particulate Matters at EP Inlet & Outlet



4.3 COMPARISON OF APPARATUS FOR COLLECTION OF PARTICULATE MATTERS

Table 5 shows the summarized comparison of various types of apparatus for collection of particulate matters.

Table-5 Comparison of Apparatus for Collection of Particulate Matters

	Cyclon Dry Type	Spray Tower (Spraying Chamber)	Venturi Scrubbers	Fluidized Bed Scrubbers	Elec. Sta. Precipitator (Dry Type)	Elec. Sta. Precipitator (Wet Type)
Method	Centrifugal Force	Scrubbling	Scrubbling	Scrubbling	Charging	Charging
System	Introduces gas into the tower tangentially, and separates the particles in the gas by centrifuge	Sprays water into the tower and induces the gas to come in contact with water at a slow speed	Injects gas into throat portion at a high speed and have it mix with water	Spraying chamber & fluidized bed are integrated; the latter, consisting of multiple bed, contains plastic balls, which forms fluidized bed, contacting with turbulent gas	Particles charged with electro statics are separated from gas by elec. force. Particles on electrodes are separated by hammering (Horizontal type)	Same as the dry type. Water flowing on electrodes collects particles (Vertical type)
Particles Collection Rate	80 – 90%	85%	98 – 99%	88 – 94%	84 – 95%	85 – 89%
Particulate Concentration at the Inlet	20 – 80 g/Nm^3	0.5–2.0g/Nm^3	3 – 10 g/Nm^3	2.2 g/Nm^3	0.3 – 7.0 g/Nm^3	0.1 g/Nm^3
Pressure Drop	40 – 110 mmH$_2$O	85–150 mmH$_2$O	350 mm H$_2$O	500 mmH$_2$O (3 fluidized bed)	10 – 65 mmH$_2$O	20 – 60 mmH$_2$O
Liquified Gas Ratio	–	8 l/m^3	24 l/m^3	5 l/m^3	–	–
Moisture Content of Gas Exhaust	–	–	–	–	16 – 42%	2 – 3%
Temperature of Exhaust Gas	–	–	–	–	240 – 300°C	15 – 20°C
Applied to	Fluidized bed	Multiple hearth/ fluidized bed	Fluidized bed	Multiple hearth	Fluidized bed	Multiple hearth
Merits & Demerits	Effective for collection of high concentration, large sized particles relative simplenple design. Efficienty drops if diameter of cyclon is made larger.	Collection absorption can be combined. Simple design. Spray holes gets plugged sometimes. Deficiency in flow may hinder adequate contact of liquid & gas.	Collection by absorption can be combined. Compact but able to process large q'ty of exexhaust gas. Highly efficient for collection of particulate matters.	Collection by absorption can be combined. Suitable for fine particles not recommended when gas speed varies.	Performance tends to vary depending upon the conditions of gas or of particulate matters.	Inlet conditions not as critical as the dry type. Suitable for extremely fine particles.

5. CONTROLLED INCINERATION AT A MULTIPLE HEARTH FURNACE

The experiments were made using a multiple hearth furnace with a capacity of 50 tons/day. The exhaust gas generated by combustion was treated by scrubbers and an electrostatic precipitator and then heated to more than 750°C by an afterburner. The resultant gas was then further processed by scrubbers and a spray tower and then discharged into the atmosphere.

In conducting the experiments, some simple renovations were made to reduce airleaks, such as closure of the air feed port at the drying zone of the furnace as well as ports for feeding shaft-cooled return air. The feeding of the dewatered cakes was maintained at the designed capacity, and the feeding of air was controlled so as to achieve the overall air ratio, while monitoring the O_2 concentration in the exhaust gas.

Experiment No.	Overall Air Ratio	Year & Date
Run I	1.73	Aug. 2, 1977
Run II	1.27	Aug. 6, 1977
Run III	1.00	Aug. 5, 1977

(1) Results

1) The results of the analysis of the dewatered sludge cake feed and ash are shown in Table 6. The cakes were dewatered with ferric chloride and lime as filter aid and then pressure filtered.
The analysis of the ash revealed that for Runs I and II, the ignition losses were 6.7 ~ 7.0%, indicating satisfactory combustion of the cakes. For Run III, however, the 13% ignition loss indicates inadequate combustion. This is because the volume reduction speed of the combustibles slowed down as the furnace condition approached the reduced atmospheric condition.

Table-6 Results of Analysis of Dewatered Cake and Ash

		Sludge	Dewatered Cakes			Ash		
		Run I	Run I	Run II	Run III	Run I	Run II*	Run III*
Moisture content	%	95.4	59.4	64.0	63.3	0.4	50.5	41.5
Fixed residue	Wt % D.B.	46.3	54.8	52.3	52.5	93.5	89.9	80.8
Volatile matter	Wt % D.B.	47.2	43.9	46.9	46.5	6.5	7	13
Fixed carbon	Wt % D.B.	6.5	1.3	0.8	1.0	0	0	0.1
Carbon	Wt % D.B.	23.16	20.03	21.33	21.52	2.30	3.49	8.66
Hydrogen	Wt % D.B.	3.75	3.20	3.22	3.14	0.22	0.87	0.88
Nitrogen	Wt % D.B.	4.01	2.84	3.18	3.17	0.02	0.07	0.50
Oxygen	Wt % D.B.	21.01	19.05	19.56	19.22	3.96	5.65	9.12
Combustibility/"S"	Wt % D.B.	1.77	0.12	0.45	0.45	0	0.02	0.04
Incombustibility/"S"	Wt & D.B.	0.96	1.72	1.71	1.71	2.82	2.88	2.51
Chlorine	Wt % D.B.	3.74	0.86	0.75	0.89	1.16	1.16	1.12
Calcium oxide	Wt % D.B.	–	28.3	19.1	19.9	44.8	47.5	38.9
Cyanogen	mg/kg D.B.	N D	7.29	14.30	14.17	N D	N D	0.56
Total chrome	mg/kg D.B.	550	510	530	580	950	1,050	950
Chromium (VI)	mg/kg D.B.	–	N D	N D	N D	6.6	N D	N D
Solubility test of chromium (VI)	mg/l	–	N D	N D	N D	N D	N D	N D
Caloric value	kcal/kg. D.B.	2,550	1,810	2,030	2,050			

* For Run II, III, Moisture was added to ash.

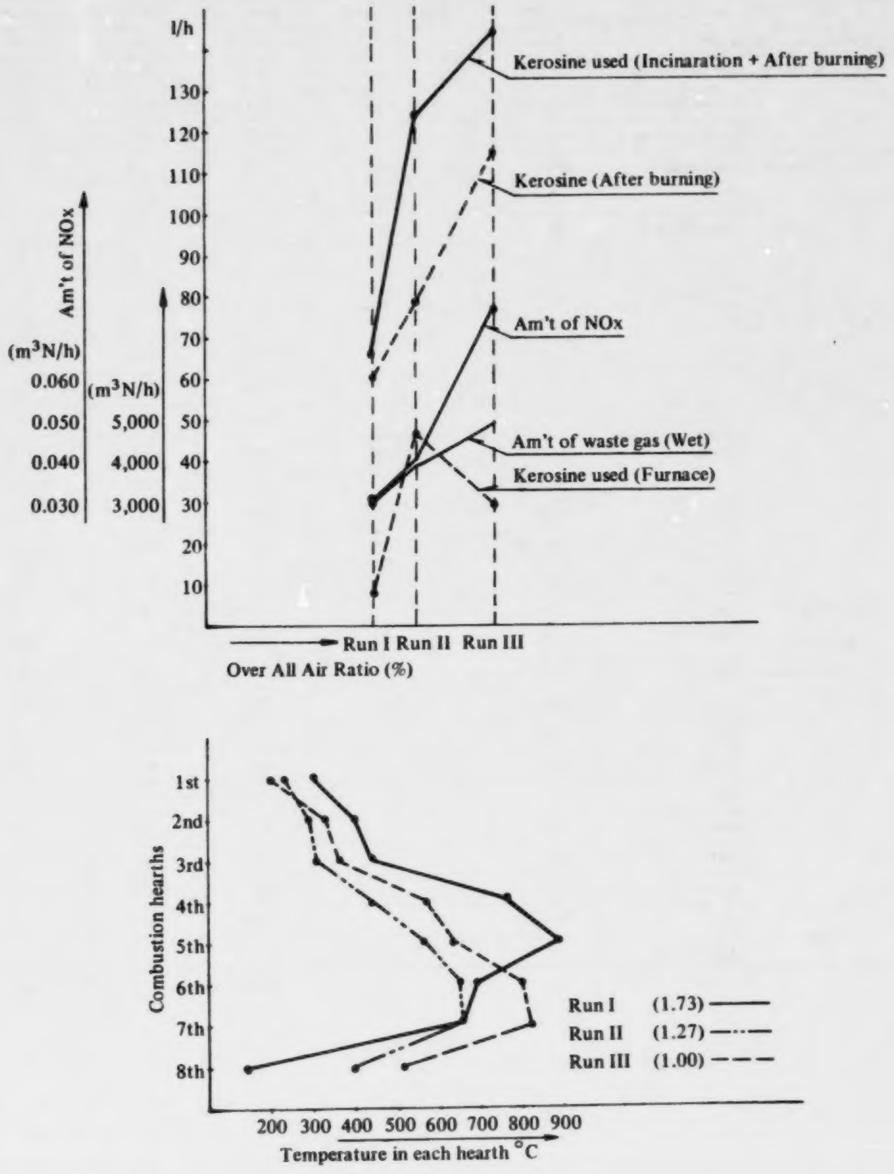
2) The overall data of the present experiments is shown in Table 7. As can be seen from the Table, by reducing the overall air feed ratio to; Run I: 1.73: 1.27, Run III: 1.00, the amount of discharged gas (wet gas) was also decreased to : Run I: 4,897 Nm³, Run II: 3,920 Nm³, Run III: 2,980 Nm³. This decrease in the discharged gas brought about a decrease in the amount of the heavy oil used for combustion of the excess gas as well as the kerosine used for re-heating, as shown in Fig. 6. Also, due to the drop in the drying speed of Run II and Run III, the temperature of the eighth hearth, normally a cooling zone, went up (Fig. 6).

Table-7 Results of Experiments

		Unit	Designed Value	Run I	Run II	Run III
Am't of Dewatered Cake		T/H	2.08	1.97	1.94	1.95
Properties of Cake	Moisture content	W%W.B.	70	59.4	64.0	64.3
	Fixed residue	W%D.B.	48	54.8	52.3	52.5
	Calorific value	kcal/kgD.B.	1,700	1,810	2,030	2,050
Am't of Kerosine Used.	Incinerate	l/H	102	29	46	7
		l/T-Cake	49.0	14.7	23.7	3.6
	After burning	l/H	240	115	78	60
		l/T-Cake	115	58	40	31
	Total	l/T-Cake	164	73.1	63.9	34.4
Am't of Comb. Air *		kg/H	–	4,060*	2,739*	1,551*
		Nm^3/H	–	3,172*	2,149*	1,212*
Over All Air Ratio		–	2.3	1.73	1.27	1.00
Properties of Exhaust Gas (At the Furnace Outlet)	Temperature	C	–	245	196	186
	Wet gas	kg/H	7,360	5,585*	4,320	3,087
		Nm^3/H	–	4,897	3,920	2,980
	Dry gas	kg/H	5,100	4,123*	2,843	1,647
		Nm^3/H	–	3,078*	2,082	1,186
	Water content *	V %	–	37.1*	46.9*	60.2*
	O_2 *	V %	–	9.0*	4.4*	0.6*
	CO_2 *	V %	–	10.3*	14.9*	18.5*
	Dust	g/Nm^3	3,000	1,979	1,547	1,666
	SOx	ppm	687	360	530	420
	NOx	ppm	–	25	19	26
	HCl_3	ppm	–	150	26, 48, 118	120
	Cl_2	ppm	–	0.2	0.07	0.2
	HC N	ppm	–	260	39	230
	NH_3	ppm	–	840	1,070	71
	Odor concentration	–	–	5,500	1.7×10^4	7.7×10^4
Ash	Am't	kg/H	325	464*	392*	418*
	Ignition loss	%	–	6.5	7*	13*

* Valuer calculated from material balance.

Fig.-6 Results of Analysis



3) Contrary to our assumption that the concentration rates of the materials contained in the gas would rise with the decrease in the amount of discharged gas, no marked changes were observed in the concentration rates of SOx and HCl. This is because large amounts of these materials are fixed in the ash and only small amounts are gasified due to the large quantity of lime used as filter aid.

According to our analysis of Run I, 89% of sulfur contained in the cake was found in the ash and only 11% had gasified. With Cl, 78% remained in the ash and 10% gasified. This means that with the decrease of discharged gas, the gasification of these materials had also decreased.

4) The NOx concentration rates, measured at the furnace outlets, were 25 ppm for Run I, 19 ppm for Run II and 26 ppm for Run III. When these values are converted on the basis of O_2 concentration = 0%, the figures become; Run I: 0.14 Nm^3/h, Run II: 0.05 Nm^3/h, and Run III: 0.03 Nm^3/h, and show a definite decrease. Attempts to check the nitrogen contents at various stages of processes for Run I revealed that the N content in the exhaust gas in the forms of nitrogen monoxide (NO), Hydrogen cyanide (HCN) and ammonia (NH_3) was 2.3 kg/h, versus 22.7 kg/h originally contained in the cake.

5) The amounts of SOx, HCl, HCN and NH_3 can be adequately dealt with through proper exhaust gas treatments such as scrubbing and absorption, satisfying the existing codes for restriction of the exhaust gas.

(2) Summary

From the fore-going, it is considered possible to operate existing incinerators at a reduced air feed rate and with proper supplemental measures such as minimization of leak air and reinforcement with insular bricks to withstand the increased temperatures at the cooling zone, thus reducing NOx formation.

The results of these experiments show that operations at the overall air ratio of 1.4 are possible, and that future operation of multiple hearth furnaces should adopt this ((controlled combustion" concept, and at the same time, try to scale down their exhaust gas treatment facilities.

6. REDUCTION OF NOx AT A FLUIDIZED BED INCINERATOR

The experiments made using a fluidized bed incinerator with a design capacity of 40 tons/day were as follows:

1) Combustion with varied cake feed and air feed quantities.
2) Combustion with varied quatities of fluidized sand.
3) Combustion with hot gas injected by a distributor using a hot wind box (start-up burner), instead of the conventional method of injection auxiliary fuel (A heavy oil) into the fluidized bed.

For the combustions described above, the concentrations of O_2 and NOx in the exhaust gas were measured.

(1) Results

1) The results of analysis of the dewatered cakes used in the experiments are shown in Table 8. No significant differences were observed in the analytic values, and it is assumed that the combusted cakes were generally of the same quality and properties. The results of analysis of the ash are shown in Table 9. The N content in the dewatered cakes averaged 2.5% per D.S, whereas N content in the combusted ash was 0.01 ~ 0.04%. From this it can be assumed that almost all of the N content in the cake had volatilized. Supposing that combustion of a D.S 1 kg generates exhaust gas of 10 Nm^3 and that all of the N

content in the cake, 2.5% per D.S, converted into NO, the resultant concentration is calculated to be approx. 4,000 ppm. The maximum NOx value by actual measurement, however, was only 99 ppm (O_2 concentration 8%). Judging from this, it is assumed that almost all of the N content had volatized in forms other than NOx. According to our experiments, the NO_2 content in the NOx was less than 10%.

Table-8 Results of Analysis of Dewatered Cakes, Sludge

			Dewatered Cake				Sludge
			52–8–23	52–8–25	52–8–29	Average	52–8–29
Moisture Content		%	71.5	72.2	75.2	73	99.0
Calorific Value		kcal/kgD.S.	2,870	2,870	2,870	2,870	3,080
Volaitle Residue		%	48.4	48.9	50.7	49.4	50.5
Fixed Carbon		%	6.2	5.9	6.6	6.23	–
Fixed Residue		%	51.6	51.1	49.3	50.6	49.5
Elementary Analysis	C	%D.S.	26.83	26.98	28.18	27.3	27.91
	H	%D.S.	4.06	4.05	4.19	4.1	4.19
	N	%D.S.	2.46	2.22	2.68	2.52	2.66
	O	%D.S.	13.32	14.08	14.03	13.8	14.14
Combustibility "S"		%D.S.	1.55	1.32	1.31	1.39	1.25
Incombustibility "S"		%D.S.	0.18	0.25	0.30	0.243	0.35
Ammonia Nitrogen		%D.S.	1.5	1.7	1.2	1.47	–
Calcium		%D.S.	0.35	0.57	0.65	0.524	–

1) Sludge obtained from Decanter's Tank
2) Dewatering: Centrifuge
3) Filter aid: Diafrock (Cationoid) 0.9~0.95 D.S./D.S.–% Added.

Table-9 Analysis of Ash

		Nitrogen	Carbon (%)	Calcium (%)
52–8–24	AM	0.01	0.01	0.06
	PM	0.04	0.58	2.08
52–8–25	AM	0.02	0.18	1.60
	PM	0.00	0.22	1.72
52–8–26	AM	0.02	0.42	1.15
52–8–27	PM	0.02	0.17	1.43
52–8–28	AM	0.01	0.73	1.54
	PM	0.02	0.54	1.84
52–8–29	AM	0.01	0.13	1.59
	PM	0.02	0.28	1.49
52–8–30	AM	0.01	0.26	2.29
	PM	0.02	0.39	1.20

2) The results of our experiments are shown in Figs. 7 to 14. It should be noted that the indicated NOx values have been converted on the basis of a 12% residual oxygen concentration rate, in keeping with the current standards for exhaust gas.

Fig.-7

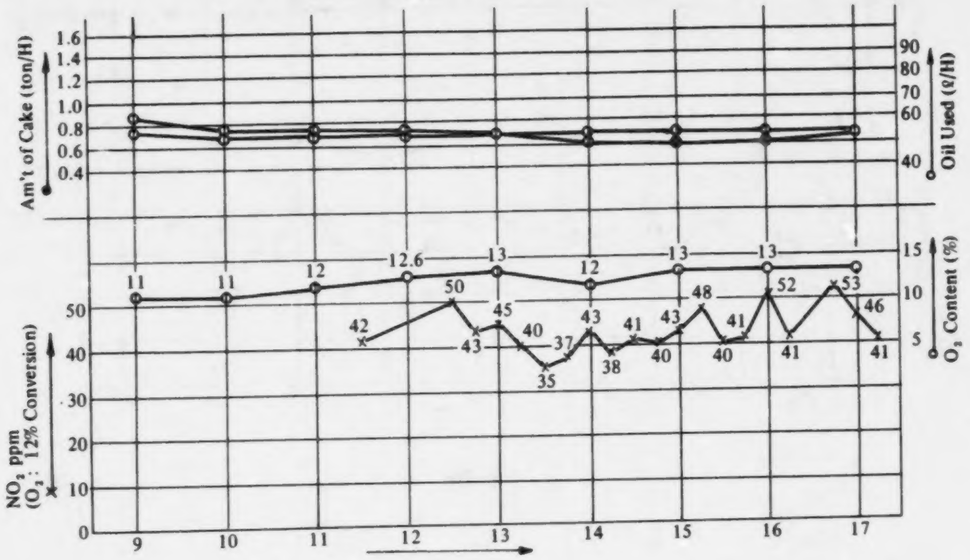


Fig.-8

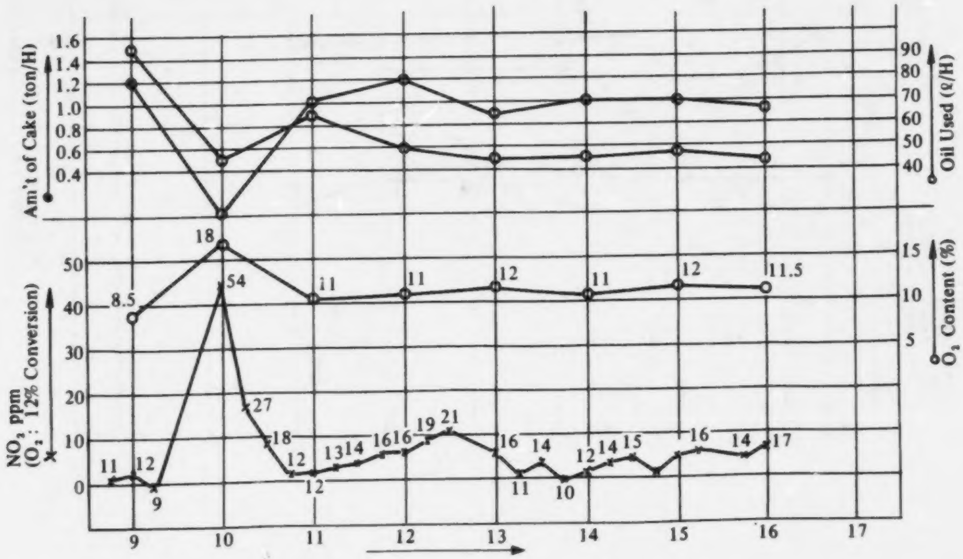


Fig.-9

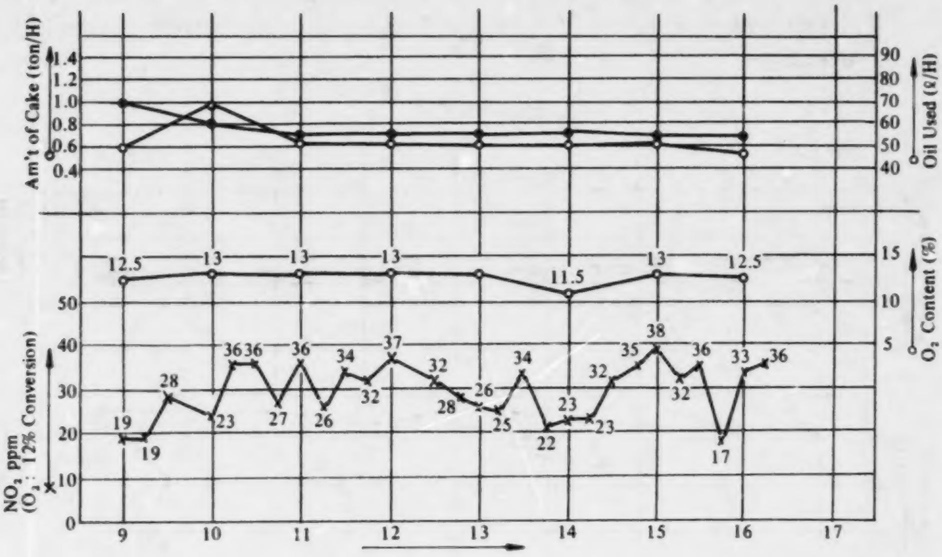

Am't of Cake (ton/H)
1.4
1.2
1.0
0.8
0.6
0.4
90
80
70
60
50
40
Oil Used (ℓ/H)
12.5
13
13
13
11.5
13
12.5
15
10
5
O₂ Content (%)
50
40
30
20
10
0
NO₂ ppm
(O₂ : 12% Conversion)
19
19
28
23
36
36
27
36
26
34
32
37
32
28
26
25
34
22
23
23
32
35
38
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Fig.-10

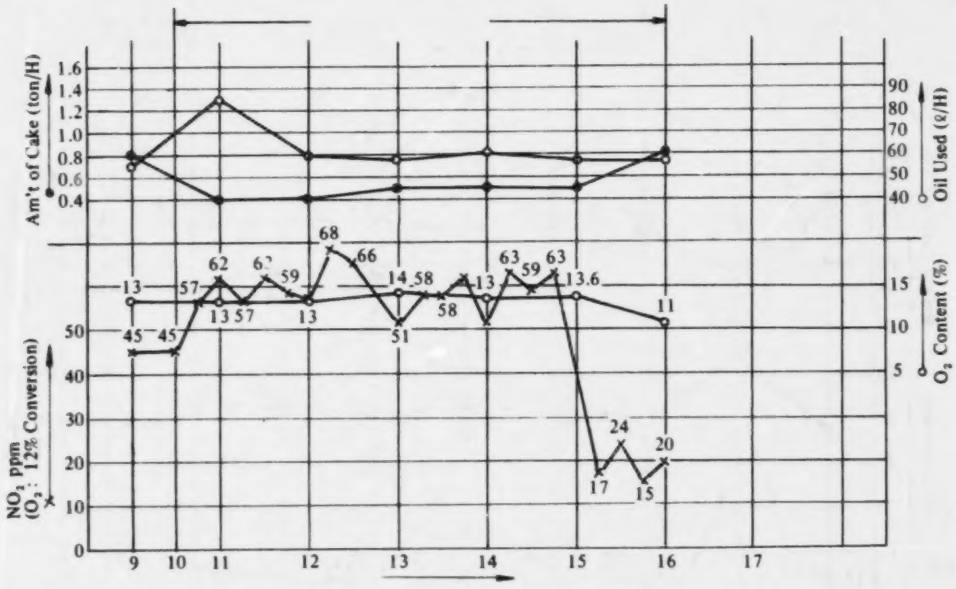
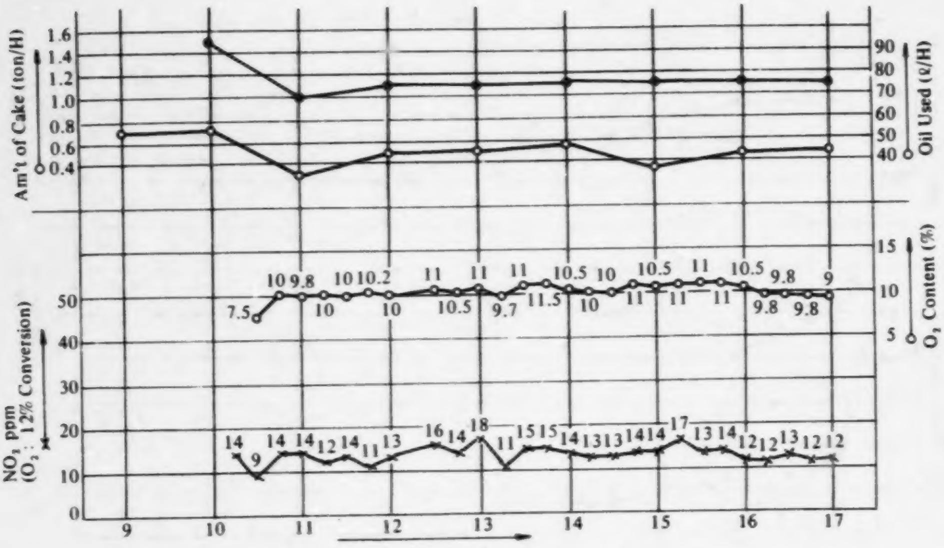

Am't of Cake (ton/H)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
90
80
70
60
50
40
Oil Used (ℓ/H)
13
57
62
13
57
62
59
13
68
66
14
58
51
58
13
63
59
63
13.6
11
45
45
17
24
15
20
15
10
5
O₂ Content (%)
50
40
30
20
10
0
NO₂ ppm
(O₂ : 12% Conversion)
9
10
11
12
13
14
15
16
17

Fig.-11



Am't of Cake (ton/H)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
Oil Used (ℓ/H)
90
80
70
60
50
40
O₂ Content (%)
15
10
5
NO₂ ppm (O₂ : 12% Conversion)
50
40
30
20
10
0
7.5 10 9.8 10 10 10.2 10 11 10.5 11 9.7 11 11.5 10.5 10 10 11 10.5 11 11 11 10.5 9.8 9.8 9.8 9
14 9 14 14 12 14 11 13 16 14 18 11 15 15 14 13 13 14 14 17 13 14 12 12 13 12 12
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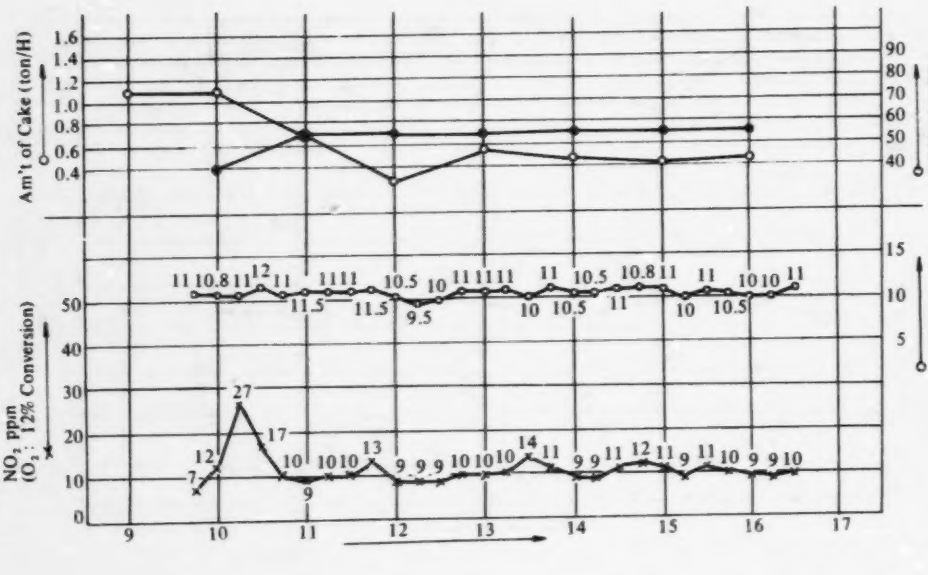
17

Fig.-12



Am't of Cake (ton/H)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
Oil Used (ℓ/H)
90
80
70
60
50
40
O₂ Content (%)
15
10
5
NO₂ ppm (O₂ : 12% Conversion)
50
40
30
20
10
0
11 10.8 11 12 11 11.5 11 11 11.5 10.5 9.5 10 11 11 11 10 11 10.5 10.5 10.8 11 11 10 11 10.5 10 10 11
7 12 27 17 10 9 10 10 13 9 9 9 10 10 10 14 11 9 9 11 12 11 9 11 10 9 9 10
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16
17

Fig.-13

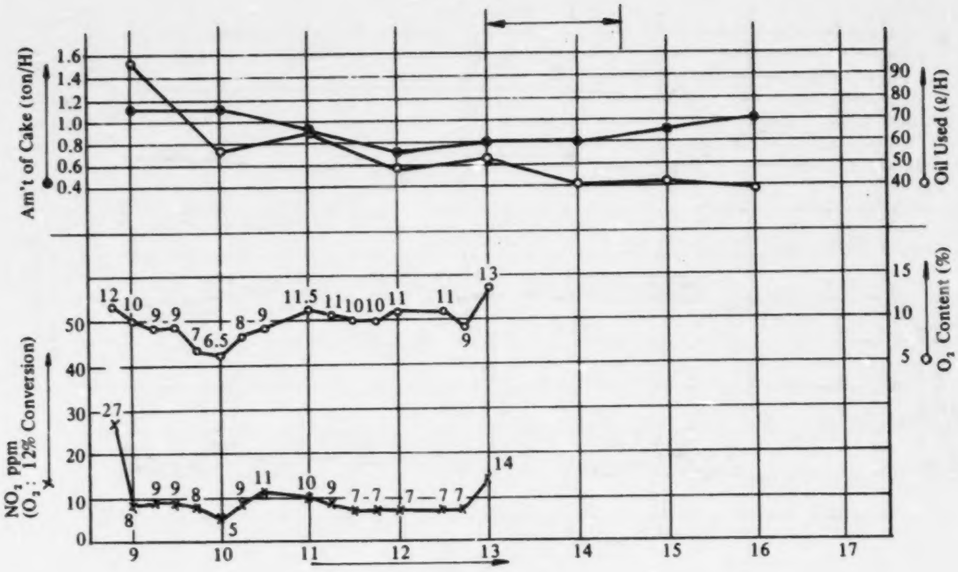
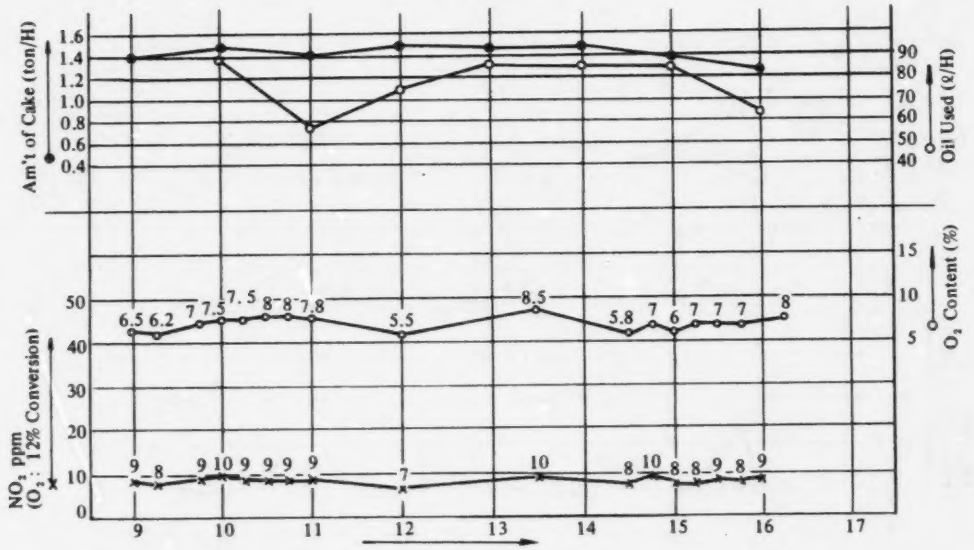


Fig.-14



Aug. 23 – Aug. 24

Prior to start up, 1.2 tons of sand were removed from the fluidized bed, thus leaving approx. 5 tons of sand. The cake feeds were 0.7 ton/h on August 23, and 1.0 ton/h on August 24. On both of these days, approximately an equal amount of combustion air was supplied to the incinerator.

The exhaust gas on August 23rd, when the cake feed was less, showed higher NOx concentration than on August 24th, when the cake feed was greater.

August 25

1.3 tons of sand were added to the incinerator to restore the bed to its normal operational condition. The cake feed was 0.7 ton/h. The amount of air feed was the same as on August 23rd. With the increased fluidized sand, the NOx concentration was lower than that of August 23rd, when the quantity of sand was less.

August 26

Instead of injecting heavy oil into the fluidized bed as in the case of normal operation, a hot wind box (start-up burner) was used for hot gas injection. The NOx concentration values were higher than the values obtained in the normal operational mode. However, since the quantity of cake feed was small, no clear relationship between this mode of operation and NOx concentration was obtained.

August 27

Combustion was made at the cake feed rate of 1.1 ton/h. The quantities of air feed and the sand were the same as on August 25th. With the increased cake feed, NOx concentration showed lower values than on August 25th, when the cake feed was less (0.7 ton/h).

August 28

0.7 ton/h of cake was combusted with the air feed restricted to the minimum, which is possible with this type of incinerator. The gap between the temperature of the fluidized sand and the free-board was great, despite the small amount of cake fed to the furnace. Presumably, this was because the combustibles, which had not combusted completely in the sand bed, combusted in the free board. The NOx concentration was lower than that of August 25th, when the cake feed was the same.

August 29

The cakes were fed at the rate of 0.7 ~ 1.1 ton/h, with the combustion air restricted to the minimum. The NOx concentration was equal to or slightly lower than that of August 28th. When the cake feed was increased to more than 1.1 ton/h, the ash, which is normally brownish in color, turned to black, indicating the pres ence of residual combustibles.

August 30

1.5 tons/h of cakes were combusted with the maximum air feed. Compared with the runs of August 25th and 27th when the amounts of air feed were the same, it revealed that the NOx concentration goes down, as the amount of cake feed is increased.

The O_2 and NOx values quoted in the foregoing were obtained by automatic monitoring. In order to ascertain the accuracy of these figures, samplings were made every day in the morning and in the afternoon. The values obtained in this manner coincided with the above-mentioned values.

These values, as well as the operational conditions from which the values were obtained are summarized in Fig. 15. For convenience of observation, lines were drawn between the measurements taken in the mornings and those taken in the afternoons. Since no analysis were conducted in the mornings of August 24th & 27th due to failure of the facilities, values for those dates were not included.

(2) Summary

1) When processing sewage sludge in fluidized bed furnaces, NOx generation can be kept to a minimum by restricting the overall air ratio. When the air feed is lower than 1.15, however, some detrimental effects from incomplete combustion take place.

2) NOx can be minimized by keeping the amount of cake feed as close as possible to, but not exceeding, the design capacity of the incinerator.

3) The incinerator used in the experiment showed a tendency that NOx increases as the amount of fluidized sand decreases.

Accordingly, in processing sewage sludge in a fluidized bed incinerator or in planning new processing facilities, it is considered advisable to maintain the overall air ratio at 1.2 for effective control of NOx.

7. REDUCTION OF NOx BY MEANS OF AMMONIA INJECTION AT A FLUIDIZED BED INCINERATOR EQUIPPED WITH PRE-DRYING HEARTHS

The fluidized bed incinerator (Capacity: 36 tons/day) used in the experiments consists of (1) the upper drying zone in the form of a multiple hearth furnace and (2) the lower combustion zone in the form of a fluidized bed. A portion of the combusted gas is led into the drying hearth, and after being used for drying, circulates back to the fluidized bed via the furnace top.

The experiments were made in the following methods.

1) Method 1: No dewatered cakes were fed and only kerosine was burnt to determine NOx emission.

2) Method 2: Dewatered cake samples from three different processing plants were combusted to investigate the resultant NOx emission.

3) Method 3: NH_3 was injected into the furnace to see if it decreases NOx emission.

(1) Results

1) Method 1: The kerosine used had N content of less than 0.01%. The results of the analysis are shown in Fig. 17. Since excess restriction of air feed raises the furnace temperature excessively, the oxygen concentration was regulated to within 10 ~ 15%, so as to maintain the furnace temperature within 800 ~ 900°C. The horizontal axis shows O_2 concentration rates at the furnace outlet and vertical axis shows NOx concentration rates as well as NOx formation per one (1) liter of kerosine.

O_2-NOx concentration curve shows that as the O_2 concentration increases, the NOx concentration shows a downward curve. This is because NOx was diluted by air. Therefore, the values were converted to the restricted standards of O_2 concentration = 12%. The result shows a nearly horizontal line. If the NOx formation per one unit of kerosine is illustrated, it also forms a nearly horizontal line. Thus, there seems to be no clear inter-relationship between the NOx formed by kerosine combustion and the O_2 concentration. It can be surmissed from the foregoing that the most of the NOx formed by kerosine in the furnace temperatures of 800 to 900°C is fuel NOx.

The Figure indicates that when O_2 concentration is high, so is the NOx concentration. This interrelationship between O_2 and NOx concentration is shown in Fig. 16.

Fig.-15

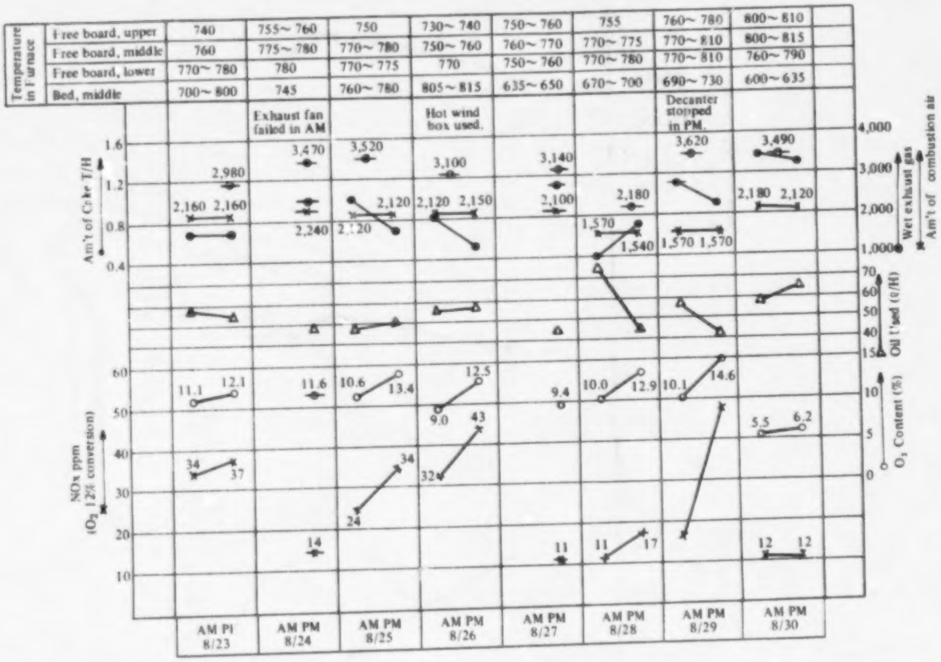


Fig.-16 O_2 Concentration and NOx Concentration

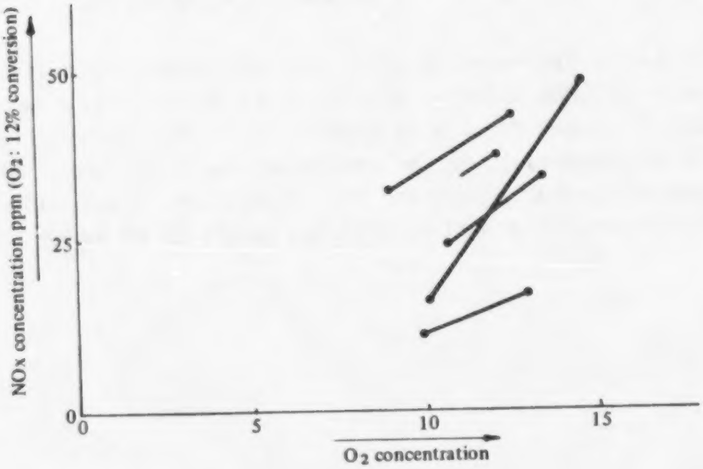
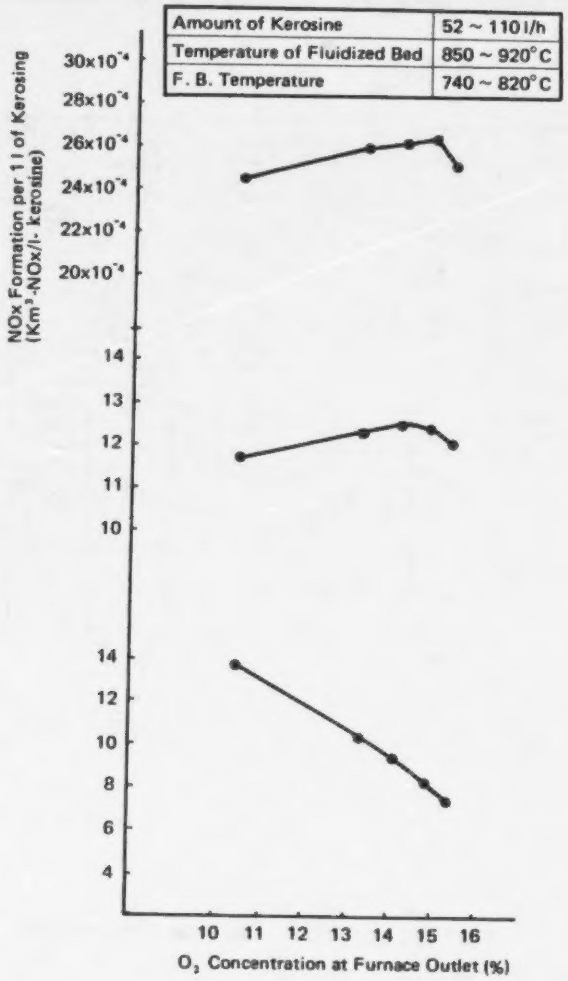


Fig.-17 NOx of Perosine Burning



Amount of Kerosine	52 ~ 110 l/h
Temperature of Fluidized Bed	850 ~ 920°C
F. B. Temperature	740 ~ 820°C

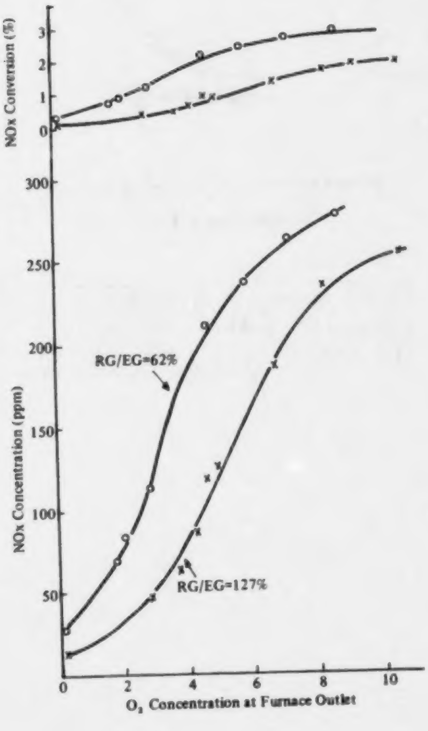
2) Method 2: The properties of the dewatered cakes used for this experiment are shown in Table 10 below. Fig. 18, 19 and 20 show the measurements taken of each of the dewatered cakes subjected to the experiments. In the Figures, the FB temperatures denote the temperatures in the free board. RG/EG show the ratio of circulating gas to the exhaust gas at the furnace outlet in percentage; RG denotes the amount of circulating gas and EG the amount of discharge gas.

Table-10 Properties of Dewatered Cakes Tested

Item \ Sample Sludge	Unit	A	B	C
Drainage System	–	Separate system	Combined system	Combined system
Digestion	–	No	No	No
Dewatering	–	Centrifuge	Filter press	Centrifuge
Moisture Content *	%	80.58	60.76	78.56
Ash Content	%	15.34	61.76	29.32
Combustibles	%	84.66	38.24	70.68
HHV	kcal/kg–D.S	4,690	1,960	3,810
LHV	kcal/kg–D.S	4,320	1,760	3,560
C	%	41.86	16.65	36.85
H	%	6.83	3.64	4.73
N *	%	1.10	0.81	1.51
S	%	1.08	0.73	1.09
Cl	%	0.22	1.10	1.70

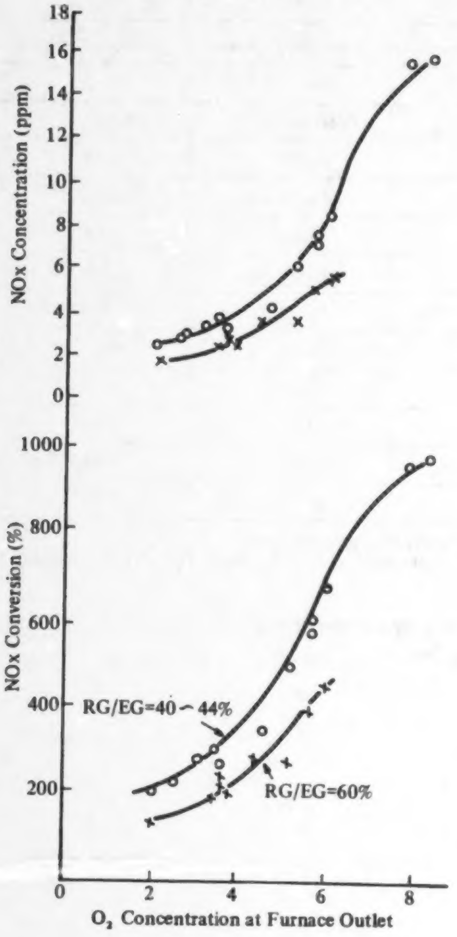
* Moisture and "N" contents are on "wet base". Others are on "dry base".
(In case of C sludge, however, "N" contents were analysed on the "dry base" and were converted to the "wet base".)

Fig.-18 NOx Formation by Combustion of A Dewatered Cakes



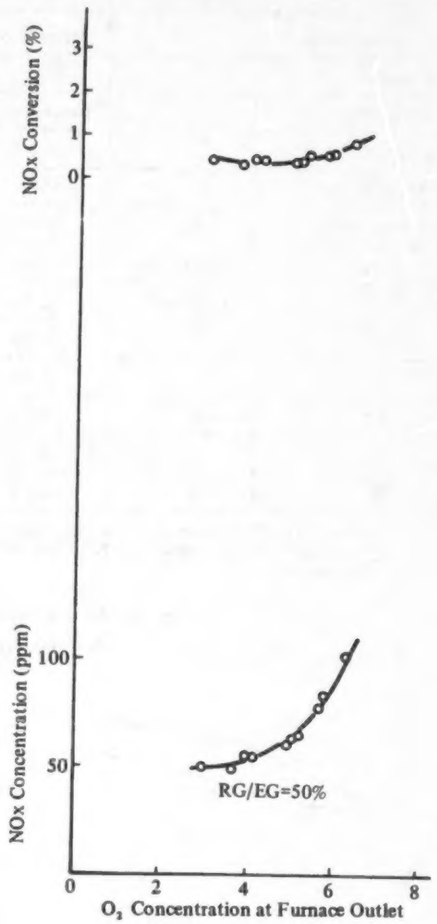
Am't Processed	1,100 kg/h
Temp. of Fluidized Bed	840～950 °C
F.B. Temp.	690～860 °C

Fig.-19 NOx Formation by Combustion of B Dewatered Cakes



Am't Processed	1,500 kg/h
Temp. of Fluidized Bed	820～890 °C
F.B. Temp.	770～840 °C

Fig.-20 NOx Formation by Combustion of C Dewatered Cakes



Am't Processed	840 kg/h
Temp. of Fluidized Bed	880～930 °C
F.B. Temp.	780～870 °C

NOx conversion rate may be defined as follows:

$$\text{NOx Conversion Rate} = \frac{\text{Amount of NOx Generated}}{\text{The Amount of NOx when all the N Content in the Sludge is Converted into NOx.}}$$

$$= \frac{\begin{matrix}\text{Quality of Dry} & & \text{NOx Concentration} \\ \text{Discharge Gas} & \times & \text{(ppm) at the} \\ (\text{Nm}^3/\text{h}) & & \text{Furnace Outlet}\end{matrix}}{\text{N Content (kg/h) in Sludge} \times \frac{22.4}{14} (\text{Nm}^3/\text{kg})} \times 100\,\%$$

A common tendency in these types of dewatered cakes is that as the O_2 concentration (air ratio) decreases, so does the NOx concentration, as can be observed from these Figures. This coincides with the conclusion of the previous survey that NOx generation is lower with a lower air ratio. This is because intermediate products such as ammonia, which are formed prior to the combustion of N contenet in the fuel, exert reducing effects to NOx under the low air feed ratio, causing NOx to lose its nitrogen oxide.

When the amount of the circulating gas is increased, it accelerates the decomposition of the residual gases such as NH_3 and HCN in the drying zone, and these residual gases in the circulating gas serve to reduce the NOx in the furnace. This phenomenon can be observed in the Figure.

The increase of the circulating gas, however, accelerates the mixing in the fluidized bed, and sometimes serves to increase the NOx. Since the dewatered cake feed for these experiments were pure sewage sludge, there was much volatilization of NH_3 and HCN in the drying zone, and it is considered that NOx reducing effect, entailed by conversion of uncombusted gas, was greater than the increase of NOx by accelerated mixing. If degested sludge were used instead of pure sludge, the results would have been different.

3) Method 3: The results of experiments using B dewatered cakes are shown in Table 11, 12 and Fig. 21. As noted in Fig. 21, the NOx concentration shows a marked decrease with the injection of NH_3, and the degree of fluctuations is also much less. Effects from the O_2 concentration are much smaller.

Fig. 22 illustrate, based upon Table 11, the relationship between the concentrations of O_2 and NOx, as distinguish by the amounts of NH_3 injected. The decrease of NOx by the NH_3 injection can be observed.

To determine the effect of the NH_3 injection in removing nitrogen oxide, the calculation was made using the following formula:

$$\text{Rate for Removal of Nitrogen Oxide} = \left(1 - \frac{\text{NOx Concentration with } NH_3 \text{ Injection}}{\text{NOx Concentration without } NH_3 \text{ Injection}}\right) \times 100\ (\%)$$

In calculating the second item on the right side of the formula, the values obtained from the actual measurements were used.

The relationships between the O_2 concentration at the furnace outlet and the rate by which the amount of nitrogen oxide is removed under a fixed amount of NH_3 injection are shown in Fig. 23. Calculation was made using a formula which determines the relationship of NOx and O_2 by the amount of NH_3 injected.

From Fig. 23, it is understood that if the O_2 concentration is more than 4%, the nitrogen oxide removal reate stays fixed, providing the amount of NH_3 injection is kept constant. This relationship in shown in the Table below:

Table-11 Results of Tests for Removal of Nigrogen Oxide (B Dewatered Cakes)

Date	A'mt of sludge kg/h	"N" content in sludge	NH_3 injection Nm /h	RG/EG %	EG(DG) Nm³/h	Cyclon Outlet		Furnace Outlet		NOx conversion %	NOx concent. w/o NH_3 injection ppm	NH_3 injection Mol Ratio	Rate for Removal of NOx	NH_3 ppm	NH_3 decomposition rate %	Remarks
						O_2 %	NOx ppm	O_2 %	NOx ppm							
12/15	1500	12.50	0	52	2381	6.3	415	5.6	436	5.19		0	0			
						4.75	290	3.3	305	3.03						
						5.3	335	4.5	352	4.19						
						4.25	255	3.4	268	3.19						Bed Temp. = 830~880°C
						5.5	325	4.7	341	4.06						F.B. Temp. = 800~840°C
						4.5	260	3.7	273	3.25						
						6.0	360	5.3	373	4.50						
			21			4.75	70	3.9	74	0.88	299	29.5	75			
						5.0	80	4.2	84	1.00	322	27.4	74			
12/23	1500	12.87	0	56	2521	6.0	420	5.3	441	5.40		0	0	4.0		Bed Temp. = 820~880°C
						6.3	450	5.6	473	5.79		0				
			10			6.0	300	5.3	315	3.86	415	9.6	24			F.B. Temp.
						4.0	140	3.2	147	1.80	229	17.3	36			
						7.0	360	6.3	378	4.63	508	7.8	66			
						4.3	160	3.5	168	2.06	257	15.4	35	2.0	99.95	
						6.5	280	5.8	294	3.60	462	8.6	36			
						7.3	370	6.6	389	4.76	536	7.4	27			
						5.3	220	4.5	231	2.83	350	11.3	34			
			15			5.0	165	4.2	173	2.12	322	18.5	46			
						3.8	115	2.9	121	1.48	210	28.3	42	28.4	99.50	
						6.2	220~250	5.5	231~263	2.83~3.22	433	13.7	47~39			*
						4.7	160	3.9	168	2.06	294	20.2	43			NOx concentration at no injection of NH_3 is calculated by next equation.
			21			6.0	150	5.3	158	1.93	415	20.1	62			
						7.2	160	6.5	168	2.06	526	15.8	68			
						3.6	85	2.7	89	1.09	191	43.6	53	4.2	99.95	NOx = 88.6 (O_2) − 50.61
						5.0	100	4.2	105	1.29	322	25.9	67			
						4.0	85	3.2	89	1.09	229	36.4	61			
						5.5	90	4.7	95	1.16	369	22.6	74			

Table-12 Furnace Temperature Dispersion and NOx (w/o NH_3 Injection)

Item \ Test Dates			
Drying fearth Temp. 1st furnace	270°C	250°C	220°C
Drying fearth Temp. 2nd furnace	405°C	425°C	425°C
Drying fearth Temp. 3rd furnace	490°C	520°C	520°C
Fluidized bed temp.	860°C	850°C	850°C
Free board temp.	780°C	840°C	840°C
Wind box temp.	263°C	300°C	285°C
Circulating gas temp.	280°C	285°C	290°C
NOx concentration in furnace (Free board, lower)	900~1,250 ppm	700~1,000 ppm	1,500~3,000 ppm
NOx concentration at the furnace outlet	350 ppm	–	300 ppm

Notes :
1. NOx concentration in furnace were measured by constant electric potential, electrolytic analysis.
2. NOx concentrations at furnace outlet were obt ained from serial records of "Luminescent Chemical Method" adjusted to furnace outlet concentration at O_2 = 4%.

Fig.-21 Changes in NH_3 Concentration by NH_3 Injection

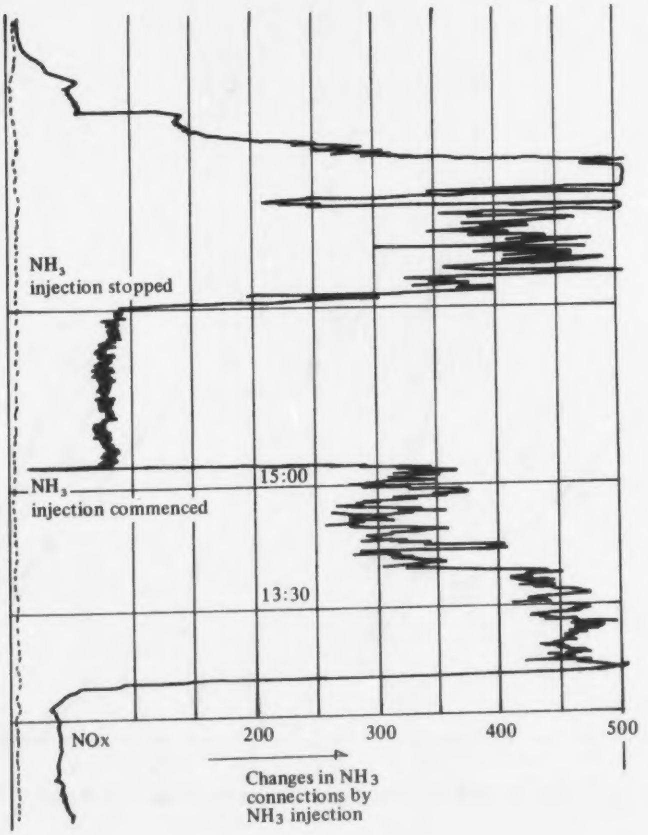
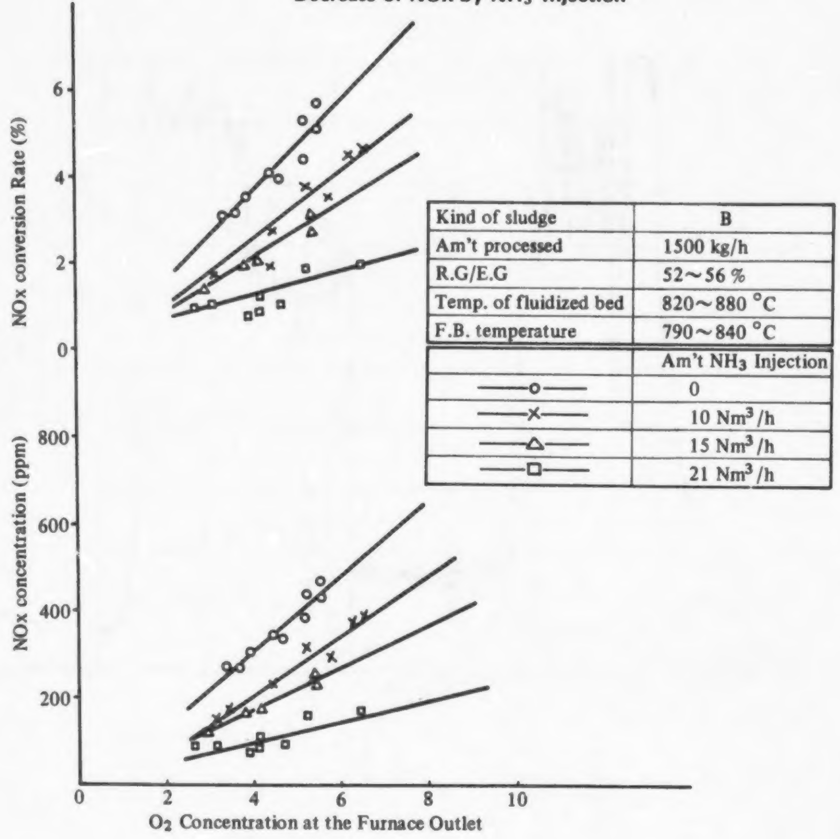


Fig.-22

Decrease of NOx by NH_3 Injection



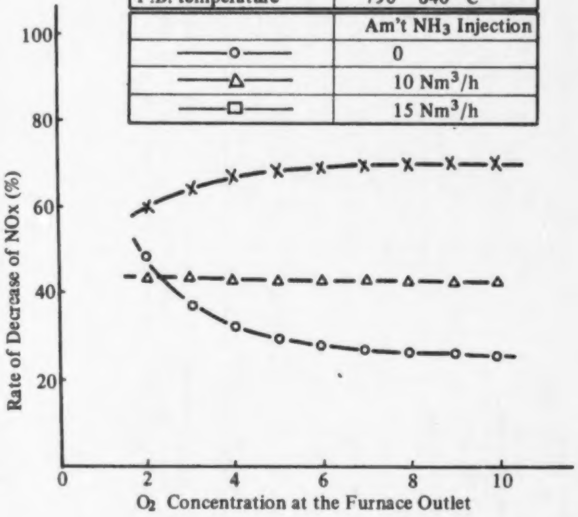
Kind of sludge	B
Am't processed	1500 kg/h
R.G/E.G	52~56 %
Temp. of fluidized bed	820~880 °C
F.B. temperature	790~840 °C

	Am't NH_3 Injection
—○—	0
—×—	10 Nm^3/h
—△—	15 Nm^3/h
—□—	21 Nm^3/h

Fig.-23 Rate of Decrease of NOx by NH_3 Injection

Kind of sludge	B
Am't processed	1500 kg/h
R.G/E.G	52~56 %
Temp. of fluidized bed	820~ 880 °C
F.B. temperature	790 840 °C

	Am't NH_3 Injection
—○—	0
—△—	10 Nm^3/h
—□—	15 Nm^3/h

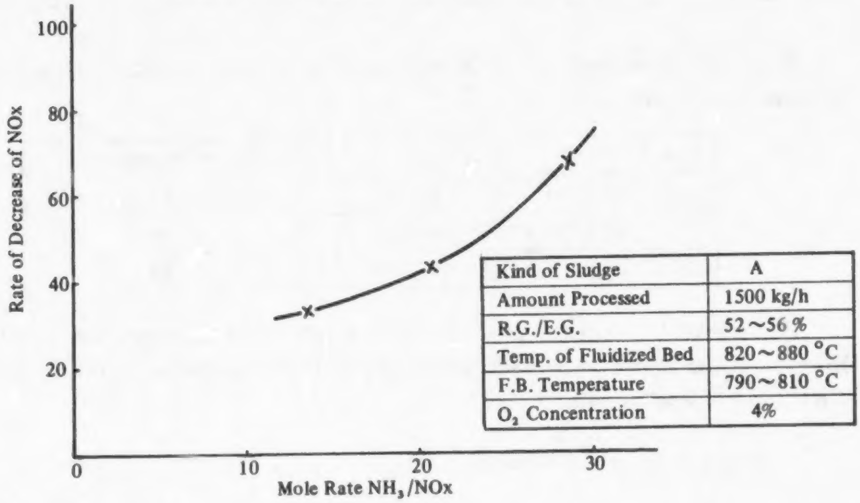


NH_3 Injection (Nm_3/h)	Rate of Removal of Nitrogen Oxide (%)
10	30
15	45
21	70

More than 99% of the NH_3 injected disappeared either by the removing of nitrogen oxide or by its self-decomposition, and the highest NH_3 concentration at the furnace outlet was 30 ppm.

Fig. 24 shows the relationship between the removal rate of nitrogen oxide and the mol ratio, with NH_3 injection. This Figure shows that the mol ratio of 29 is required to obtain the removal rate of 70%, and the mol ratio of 23 to achieve 50%. If the reaction between NH_3 and NO is improved, however, it is believed that further reduction of the mol ratio would be possible. This indicates the need to explore the optimum injection method and the need to protect the NH_3 from decomposing itself before it is used for decompostion of nitrogen oxide. The optimum temperature for this process is between 800 ~ 1,000° C. The temperatures used for the present experiments were between 800 ~ 900° C.

Fig.-24 Mol Ratio NOx Removal Rate



Kind of Sludge	A
Amount Processed	1500 kg/h
R.G./E.G.	52 ~56 %
Temp. of Fluidized Bed	820~880 °C
F.B. Temperature	790~810 °C
O_2 Concentration	4%

(2) Operating Cost

To Predict the increase in the operating cost by NH_3 injection, the volume increase at the time of injection was calculated at the cake feed rate of 1.5 ton/h against the estimated operating cost with no NH_3 injection. The operating cost with no NH_3 injection, based upon our past actual measurements, are as follows:

	Quality Used	Unit Cost	Cost (¥/h)	Cost per 1 t Cake (¥)	Ratio (%)
Kerosine	110l/h	¥34/l	3,740	2,493	64
Elec. Power	120 kw	¥15.5/kwh	1,860	1,240	32
Caustic Soda	1 kg/h	¥65/kg	65	43	1
Ind. Water	3.5 ton/h	¥15/ton	52	35	1
Fluid. Bed S.	7.5 kg/h	¥16/kg	120	80	2
Total			5,837	3,891	100

Condition: B Sludge, Water Content, 60.76%
Low Calorific Value: 1,760 kcal/kg – D.S

The coast increase by the NH_3 injection is estimated as follows:

Case	NH_3 Injection Mol Ratio	Rate of Removal of Nitrogen Oxide	NH_3 Nm^3/h	Inject. kg/h	NH_3 U.C (¥/kg)	NH_3 Cost (¥/h)
1	13.6	33	10	7.6	90	684
2	20.4	44	15	11.4	90	1,026
3	28.5	67	21	15.9	90	1,431

From the foregoing, the total cost and the rate of increase for each case are estimated as follows:

Case	Total Cost	Rate of Increase (%)
1	¥5,837 + ¥687 = ¥6,521	12
2	¥5,837 + ¥1,026 = ¥6,863	18
3	¥5,837 + ¥1,431 = ¥7,267	25

To achieve 67% nitrogen oxide removal rate, 25% cost increase is necessary. However, if the injection is made under an optimum condition and if the mol ratio of NH_3 can be kept around 3;

$$15.9\ \text{kg/h} \times \frac{3}{28.5} = 1.67\ \text{kg/h}$$

$$1.67\ \text{kg/h} \times ¥90/\text{kg} = ¥150/\text{h}$$

$$¥150/\text{h} + ¥5{,}837/\text{h} \times 100 = 2.6\%$$

Thus, adequate cost effectiveness can be obtained.

(3) Summary

1) Combustion of Kerosine Only

a) The amount of NOx generated by burning only kerosine stays the same regardless of the air-feed ratio, as follows:
NOx concentration (O_2 = 12% conversion) = 12 ppm
The amount of NOx per 1 liter of kerosine - 2.6×10^{-4} Nm^3

2) Combustion of Sludge

   a) For the three types of dewatered cakes combusted, the NOx concentration at 4% O_2 concentration (O_2 =12% conversion) ranged from 30 to 170 ppm, and the NOx conversion rate between 0.5 ~ 5%.

   b) The factor which exerts the greatest influence over NOx generation is the air ratio. As the air ratio increases, NOx also increases. Neither the amount of sludge feed nor the furnace temperature has a noticeable effect on NOx.

   c) The sludge with a large N content does not necessarily have a high NOx conversion rate.

   d) NOx tends to decrease as the amount of circulating gas is increased. This is due to the increased volatilization of NH_3 and HCN in the drying zone, which accelerates the NOx reduction function.

3) Non-catalytic removal of nitrogen oxide by NH_3 injection.

   a) It was confirmed that NOx can be greatly reduced by injecting NH_3 into the sludge incineration. It has therefore been proven that the "non-catalytic removal of nitrogen oxide by NH_3 injection" can also be used with the sludge incineration.

   b) More than 99% of the NH_3 injected was consumed either by the removal of nitrogen oxide or by its self-decomposition, and the highest NH_3 concentration at the furnace outlet was 30 ppm.

   c) To achieve 70% and 50% removal of nitrogen oxide, the mol ratios of 29 and 23 are required respectively. Judging from the actual performance of the boiler, however, operation at a lower mol ratio would be possible, providing the injection method of NH_3 is optimized.

8. REFERNCE MATERIALS

(1) Iidani "Apparatus for Collection of Parteculate Matters" 1972

(2) Kato "Journal of the J.S.M.E." Vol. 42, No. 582, 1976

(3) Kurosawa "Problems by NOx from Stationary Sources" ppm, Vol. 4, No. 2, 1973

(4) Toba "Experimental Investigation of NOx Formation in the Fluidized-bed Coal Combustion" Journal of the Fuel Society Vol. 56, No. 604, 1977

(5) R.C. Hoke, L.A. Ruth, H. Show "Combustion & Desulfurization of Coal in Fluidized-bed of Limestone" Combustion Jan. 1975

(6) Kasaoka "Reduction Removal of Nitrigen Oxides by Non-catalytic Method" Chemfactory Vol. 20, No. 7, 1976

(7) Yamanaka "Practicals of Facilitie of Reduction Removal of Nitrigen Oxides by Non-catalytic Method" Heat Control and Publio Niusance Vol 29, No. 2, 1976

(8) "Away to lower NOx in Utility Boiler" Environmental Science & Technology Vol. 11, No. 3, March 1977

CHAPTER 5

PILOT PLANT STUDY OF TANNARY WASTE TREATMENT BY OXYGEN ACTIVATED SLUDGE PROCESS

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CHAPTER 5

PILOT PLANT STUDY OF TANNARY WASTE TREATMENT BY OXYGEN ACTIVATED SLUDGE PROCESS

1. INTRODUCTION

From old days in Hyogo Prefecture, centering Himeji city, a large number of tanneries have clustered around specific areas. A total of 9 such tannery communities are found in the prefecture in Himeji City, Kawanishi city, Tatsuno city and Taishi town. In Himeji city, there are the Takagi area, Shigo and Gochaku areas, Kawanishi area, Fukui area and Jippo area with 234, 99, 37, 20 and 9 tanneries respectively.

From early days, Himeji was noted for its production of 'Himeji Leather', a white leather finished by its unique tanning process. However, with the passing of time, vegetable tanning was introduced and also chromic tanning. And today, the majority of tanneries employ the vegetable-chromic tanning.

The majority of the tanneries are either small-sized enterprises or privately-run plants. The average number of workers at an enterprise level is 32.6 and 5.6 at a privately run level. Owing to the fact that it is privately-run plant galore in the city, the average number of workers of all plants in Himeji is as low as 7.5.

The manufacturing process of tanneries in Hyogo prefecture including Himeji city can be divided as follows: (a) preparing, (b) tanning, and (c) finishing. Fig. 1.1 shows the process in detail. Tanneries use a great amount of industrial water and about 70 percent of the water consumed is used up at the preparing stage. Table 1.1 shows the amount of water used at a chromic tannery plant processing about 30 kilograms of raw leather (said capacity), and also the wastewater quality and organic and other pollutant loads. Waste from chromic tanneries contains a large amount of suspended solids, organic matters and tri-valent chromium.

Fig.-1.1 Manufacturing Flow-chart of Chromic Tannery

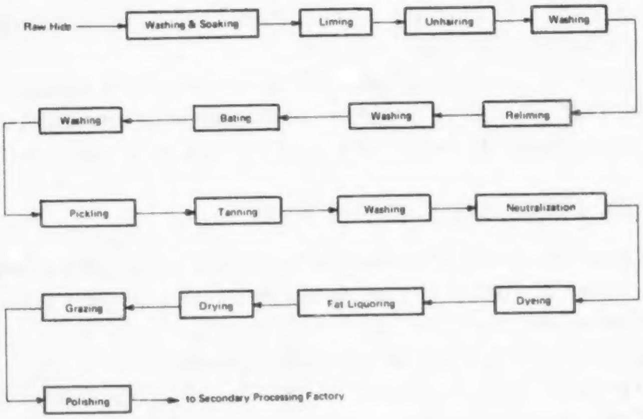


Table-1.1 Composition of Waste Water from Chromic Tannery held 300 kg Raw Hide Treatment Capacity

Items \ Processing		Washing & Soaking		Liming & Un-hairing	Washing		Reliming	Washing	Bating	Washing	Pickling	Chromic Tanning
		Pre-paration	Main Waste		1st & 2nd Washing	3rd Washing						
Weight of Hide	(kg)	330	—	—	—	270	—	—	—	—	—	—
Water Consumption	(ℓ)	1,650	1,650	1,650	3,300	1,350	1,350	4,450	810	4,450	270	270
Wastewater Discharged	(ℓ)	1,530	1,620	1,610	3,260	1,320	1,330	4,500	800	4,410	320	260
pH		6.3	7.1	12.6	12.4	12.3	12.7	12.5	9.3	8.9	3.0	3.3
Total Solids	(mg/ℓ)	49,710	12,420	25,708	3,873	2,364	5,687	817	3,882	572	54,193	68,503
Dissolved Solids	(mg/ℓ)	47,550	12,070	22,960	3,549	2,057	4,980	638	3,172	520	53,953	68,443
Suspended Solids	(mg/ℓ)	2,515	350	2,748	324	307	707	179	710	52	240	60
Ash	(mg/ℓ)	46,960	11,928	15,300	1,343	1,451	2,714	419	2,447	287	50,280	52,003
Ignition Loss	(mg/ℓ)	2,750	493	10,408	1,530	913	2,973	398	1,435	285	3,913	16,500
Total Nitrogen	(mg/ℓ)	552	111	1,670	179	132	204	38	760	73	191	75
Dissolved Total Nitrogen	(mg/ℓ)	403	83	1,554	164	116	182	19	712	69	183	67
Chloride	(mg/ℓ)	26,709	4,628	2,422	843	592	628	135	771	188	20,824	9,903
Iodine Consumption	(mg/ℓ)	43	14	3,184	232	54	405	34	16	—	2	—
Calcium	(mg/ℓ)	70	38	2,470	396	221	1,445	160	243	52	731	157
CODMn	(mg/ℓ)	8,881	890	13,007	1,309	1,148	2,162	201	419	27	1,866	3,899
Cr_2O_3	(mg/ℓ)											7,071

Each city having such groups of tanneries within its administrative area in Hyogo Prefecture has already constructed pre-treatment facility and is exerting efforts to treat waste by sedimentation. This pretreatment facility, in most cases, is composed of grit chamber, screens (coarse and fine screens), pumps (screw pump), sedimentation tanks provided with track-mounted sludge scraper (some with flocculation basins), sludge thickeners, sludge heat treatment units (heating the sludge between 90 and 100 degrees centigrade to denaturate the protein contents and to facilitate dehydration), vacuum filters and deodorization installation to remove odor.

The industrial waste from tanneries is conveyed to the pre-treatment facility via covered concrete rectangular sewers or concrete pipes by gravity. Therefore, it is necessary to prevent organic solids from depositing as possible in the sewers. And to do this, a rotary fine screen is installed in each factory to receive screened effluent into the sewer.

The existing pre-treatment facilities were designed to remove about 70 percent of suspended solids, but in future further studies will be made to increase the design removal rate of suspended solids.

The sedimentation tank effluent of the pre-treatment facility contains 600 – 800 mg/l of BOD, therefore a biological treatment is envisaged. This study was conducted to determine its design criteria of the biological treatment facilities.

2. STUDIES CONDUCTED IN THE PAST

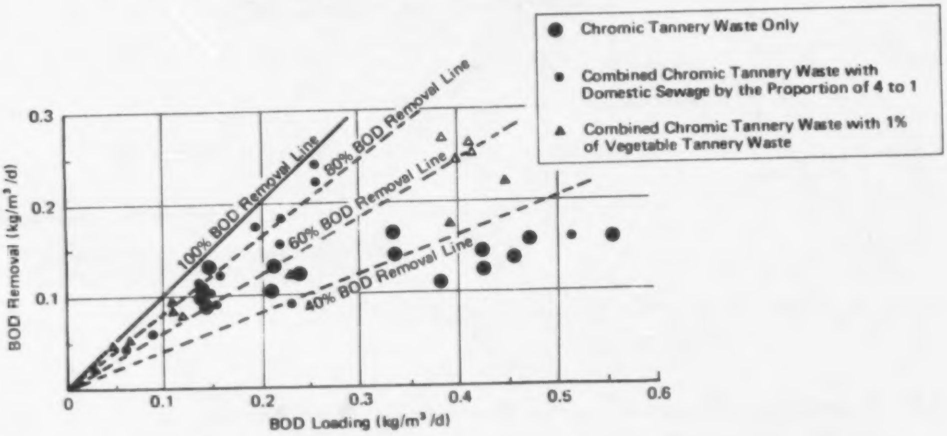
Several experiments were conducted in the past concerning biological treatment of tannary wastes in Himeji city on laboratory scale and pilot plant scale. Results are summarized as follows;

(1) The experiment of low-rate trickling filter process

This experiment was conducted in 1968 – 69 at the Public Works Research Institute of the Ministry of Construction. The experiment was conducted in a constant temperature room of 20°C by using a specially prepared laboratory scale trickling filter apparatus. The influent used in the experiment was (i) chromic tannery waste only, (ii) mixture of chromic tannery waste and domestic sewage at a ratio of 4:1, and (iii) mixture of chromic tannery waste and 1 percent vegetable tannery wastewater.

With regards experiment results, although the pH of the influent changed on the alkali side between 9 and 12, the pH of the effluent was stable between 8 and 8.5 indicating the existence of a strong buffer action. In other words, it became obvious that the treatment was possible without any pH control. The BOD removal against BOD Load and the removed BOD are as shown in Fig. 2.1. The BOD removal when the BOD Load was below 0.05 kg/m³.d and when below 0.1 kg/m³.d was over 80 percent and between 70 and 80 percent, respectively. However, in the experiment using chromic tannery waste only as influent, the BOD removal did not exceed 0.15kg/m³.d. The experiment for mixing chromic tannery waste and domestic sewage, have shown the most stable treatment results.

Fig.-2.1 BOD Loading vs. BOD Removal at the Laboratory Test of Low-rate Trickling Filter



(2) Aerated Lagoon Experiment

This experiment was conducted at the same time with the above mentioned low-rate trickling filter process at the Public Works Research Institute. The influent used in the experiment was the same as those used in the low-rate trickling filter process except for increasing the mixing rate from 1 percent to 5 percent of vegetable tannery waste.

The aerated lagoon test on a laboratory scale was conducted in the constant temperature room at 10°, 20° and 30° centigrades, respectively. The experiment results as shown in Fig. 2.2 have shown that the BOD removal was greatly affected by changes in temperature, and, that the BOD removal dropped markedly in low water temperatures. Also in the experiment using influent mixed with chromic tannery waste and domestic sewage, as shown in Fig. 2.3, the best treatment results were obtained.

Fig.-2.2 Detention Time vs. Remaining BOD at the Laboratory Test of Aerated Lagoon (Chromic Tannery Waste Only)

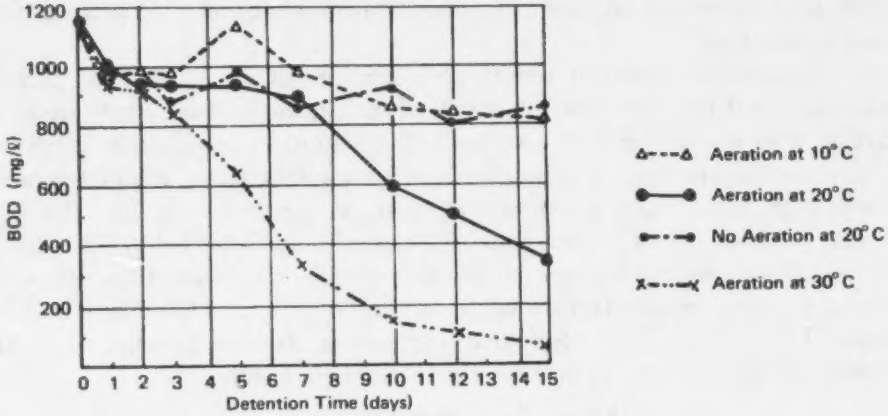
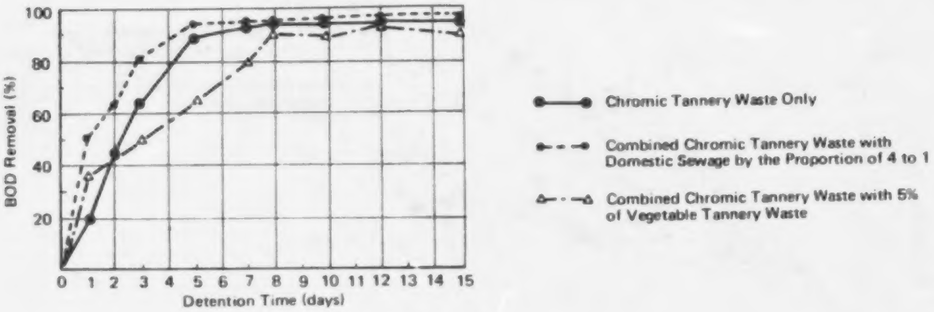


Fig.-2.3 Detention Time vs. BOD Removal at the Laboratory Test of Aerated Lagoon (20°C)



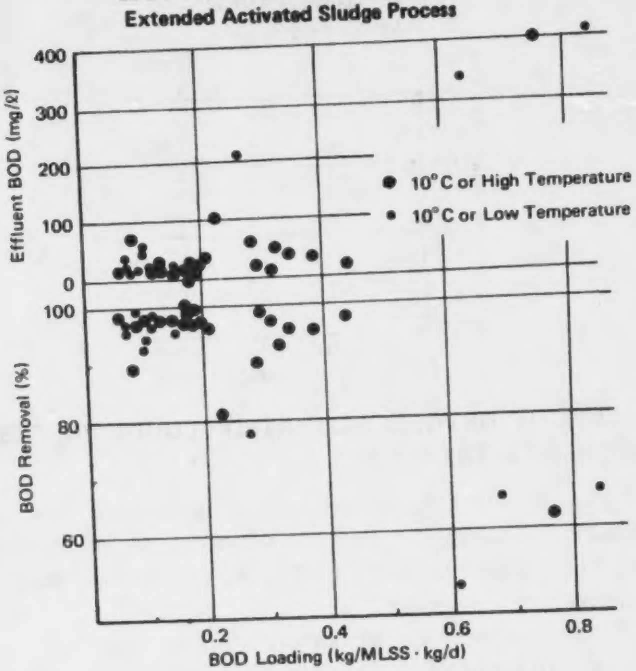
(3) Experiment of Activated Sludge Process in Extended Aeration

In this experiment, influent containing chromic tannery waste only was used. And the test was held outdoors employing a small scale pilot plant. At that time, the pretreatment facility in Takagi area was in operation. Therefore, the effluent from this pretreatment facility (plain sedimentation) was used for the activated sludge treatment experiment.

The average quality of pretreated wastewater was BOD 782 mg/l, COD_{Mn} 311 mg/l, SS 120 mg/l, total chromium 7.1 mg/l and pH 8.0 – 11.8. pH was controlled in advance by dosing sulfiric acid. The aeration tank was operated by the completely mixing mode at MLSS 3,000 – 4,000 mg/l. And the experiment was conducted from fall to spring. Fig. 2.4 shows the relations between the BOD Load and treatment efficiency.

Comparatively favorable treatment efficiency was obtained when the BOD Load was under 0.1 kg/MISS.kg/d. But the fluctuation in effluent quality was inevitable and sludge bulking phenomenom was observed.

Fig.-2.4 BOD Loading vs. Effluent BOD and BOD Removal at the Small Scale Pilot Plant Test of Extended Activated Sludge Process

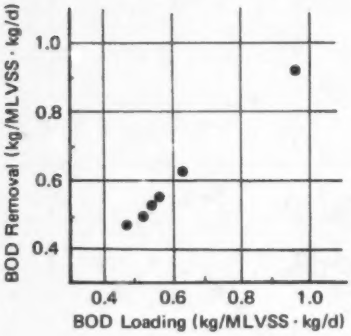


(4) Preliminary Experiment of Pure Oxygen Activated Sludge Process

This experiment, similar to the experiment in (3), used effluent from the pretreatment facility of Takagi area and was conducted at a pilot plant mounted on a trailer. The water quality of the influent was the same as (3) but the experiment was conducted only limited period of between summer and fall. The experiment was held as follow: Aeration time: 3.5 – 6 hours; and MLSS 6,500 – 7,600 mg/l. There was no pH control in this experiment.

Part of the experiment results is shown in Fig. 2.5. As the figure shows, when the BOD Load was 0.37 – 1.07 kg/MLVSS. kg/d, the BOD removal rate was over 97 percent, thus a linear relationship between BOD Load and BOD removal was obtained. Furthermore, oxygen consumption in the aeration tank was roughly the same as BOD removal. The total chromium in the activated sludge changed greatly from 510 to 31,000 mg/kg, but when SRT was kept long, chrome accumulated in the activated sludge and have shown a tendency to affect biological activity.

Fig.-2.5 BOD Loading vs BOD Removal at the Pilot Plant of Pure Oxygen Activated Sludge Process



3. APPLICATION OF OXYGEN ACTIVATED SLUDGE PROCESS FOR TANNARY WASTE TREATMENT

From the experiments held in the past, it has become clear that the most favorable treatment efficiency can be expected from the pure oxygen activated sludge process. When introducing the pure oxygen activated sludge process, the following advantages can be expected:

i) pH control is not necessary for the tannery waste treatment.

ii) As it is possible to maintain MLSS in the aeration tank at a high concentration, it is possible to decrease in aerator capacity or shortening of aeration time.

iii) There is a possibility of preventing the sludge bulking phenomenon.

iv) The oxygen supply deficit that is likely to occur in high organic waste treatment can be prevented.

Of the reasons cited, it was decided most appropriate to introduce the pure oxygen activated sludge process in treating tannery waste. Also, as Himeji city is planning to construct a coastal station for LNG (Liquid Natural Gas) transported from abroad in the near future, there is a great possibility of obtaining cheap oxygen produced at this station and conveyed through a pipe line. That was another reason why oxygen activated sludge process was adopted.

However, in the past oxygen activated sludge process experiments, the test periods were as short as four months and were held at summer time. Therefore, at this study, the following problems must be served:

a) Even during day time, the fluctuation of tannery waste quantity and quality is great. However, at night time and on Sundays, there is virtually no wastewater coming in.

Therefore, it was requested to determine whether the waste from tanneries should be treated with diurnal variation or equalized condition.

b) Experiment results of through the four seasons by oxygen activated sludge process were not available.

Therefore, it was required to conduct experiments in winter to obtain necessary data to design facilities.

c) The effluent from the pretreatment facilities at Takagi area and Shigo & Gochaku areas are expected to be transported to the Himeji Tobu Wastewater Treatment Plant located near the coast by an exclusive pipe line extending about 6 kilometers.

The effluent from these pretreatment facilities totals about 28,000 m^3/d. It was required to determine whether it is better to separate this effluent from other domestic sewage for exclusive treatment or should be combined with domestic sewage and others at a ratio of 1:1.

d) The effluent standard of BOD 20 mg/l (maximum: 25 mg/l) is to be applied to the enfluent from the Himeji Tobu Wastewater Treatment Plant. Also in the Seto Inland Sea water areas, the total emission of wastewater load by COD_{Mn} is to be regulated after 5 years. Presently, the total dischargeable load by COD_{Mn} from this wastewater treatment plant is undecided. Therefore, it is necessary to consider the removal of COD_{Mn} when conducting this experiment.

e) In Japan, three types of oxygen activated sludge processes are put on the market. They are the UNOX system, the MAROX system and the EBARA-Infilco system. It is important to study thoroughly the merits and demerits of each system and select the most economical and effective system for operation when designing the actual facilities of the Himeji Tobu Wastewater Treatment Plant.

To solve the problems mentioned above, pilot plants of the three companies were installed in the compound of the Takagi Pretreatment Facility to conduct 15 month experiments. The pilot plants of the three companies were each operated by engineers from their own. Also, city officials and members of private consulting firms cross checked the water quality data and the operating data to assure fairness in giving engineering evaluations of the three systems.

Himeji city, in order to instruct, supervise and evaluate the experiments at the three pilot plants, has organized the Engineering Evaluation Committee of Tanneries Waste Treatment Experiment in Himeji. The members of this committee included officials from the Public Works Research Institute of the Ministry of Construction, Hyogo Prefectural Industrial Research Laboratory, Public Works Department of Hyogo Prefectural Government and Environmental Department of Hyogo Prefectural Government.

4. EXPERIMENT PROGRAM AND PROCEDURE

As the experiments of the pilot plants of three companies were conducted to obtain fundamental data and to confirm the problems with regards operation and maintenance to design the Tobu wastewater treatment plant, the following condition was given:

i) Experiment of constant feed of tannery waste only
ii) Experiment of varied feed of tannery waste only
iii) Experiment of mixed feed of varied tannery waste and constant domestic sewage.

With regards each experiment listed above, the self-active operation term of which the operation condition is designated by free judgement of each company (seek operation condition during this period to secure target effluent quality) and the evaluating operation term of which the treatment results of the designated operation condition are evaluated.

The aforementioned experiments i) and ii) were held twice in summer and winter, and the evaluating operation term for each season was instituted.

In experiment i), the pretreatment effluent was supplied by a constant feed pump. In experiments ii) and iii), the typical influent flow pattern was given from the measured influent flow results. Furthermore, in experiment iii), without changing the flow pattern in experiment ii), domestic sewage equivalent to 1/2 of the total volume was to be fed constantly. Fig. 4.1 and Fig. 4.2 respectively show the flow patterns used in experiments ii) and iii).

Fig.-4.1 Fluctuation Flow Pattern of Chromic Tanneries Waste Only

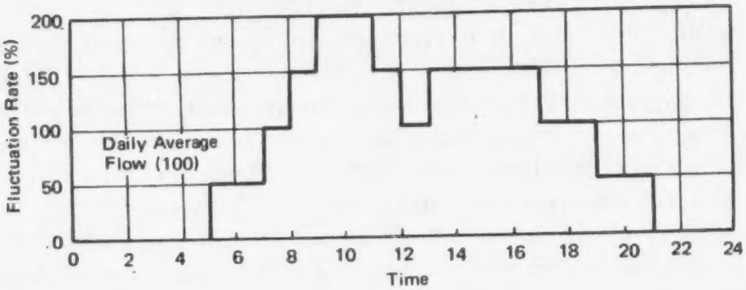
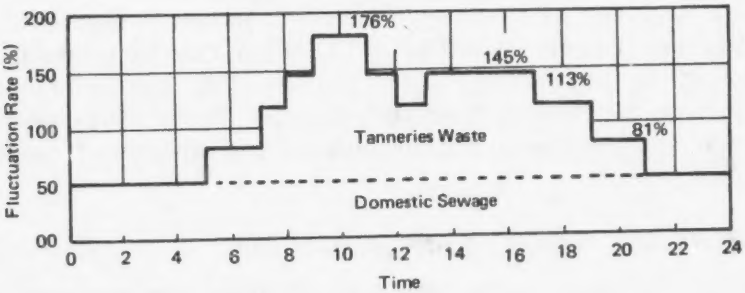
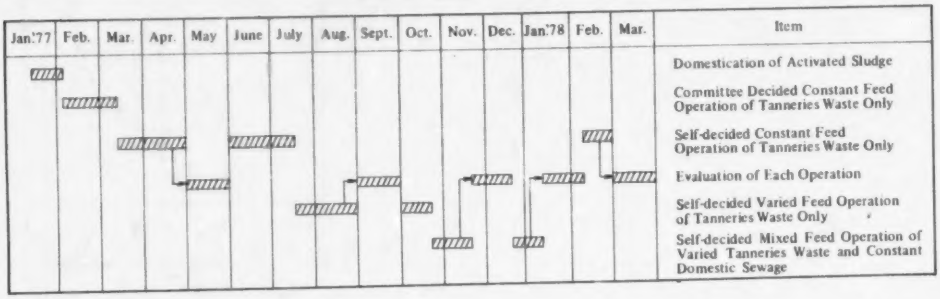


Fig.-4.2 Fluctuation Flow Pattern of Mixed Chromic Tanneries Waste with Domestic Sewage



Prior to the commencement of the experiments, experiment plan for the next 15 months was compiled. But as the start of the experiments was in winter and too much time was consumed on the aclimation of activated sludge, the initial plan was postponed. As a result, the experiment was held under experimental conditions as shown in Fig. 4.3.

Fig.-4.3 Pilot Plant Experimental Plan for Chromic Tanneries Waste Treatment



Remarks:

1) Experimental condition for the Committee decided constant feed operation of tanneries waste only was detention time of six hours and mixed liquor suspended solids of 6,000 mg per litre in the aeration tanks.

2) Experimental condition for evalution of each operation was decided with understanding of the Committee on the basis of the proposal of each manufacture. It was the major premise that the effluent was maintained both BOD of 20 mg per litre or less and CODMn of 50 mg per litre or less.

Of the operating conditions for pilot plants shown in Fig. 4.3, the standard operating conditions centering in February of 1977 were: detention time 6 hours, and MLSS over 6,000 mg/l to operate the pilot plants. Also during the evaluating operating period, each company, based on the results of the self-active operating period, was asked to submit its operating conditions to the Committee for approval. In this instance, the BOD of the effluent was set within 20 mg/l and COD_{Mn} within 50 mg/l as a target.

Of the three companies, Ataka Kogyo Co. which owns the MAROX system submitted a proposal in July 1977 to conduct the oxygen activated sludge process experiment in two stage process to the Committee. And thus, the MAROX system has employed the two stage process ever since August of 1977.

In Fig. 4.4 (a) and (b), the pilot plant of MAROX system, in Fig. 4.5 the pilot plant of EBARA-Infilco system and in Fig. 4.6 the pilot plant of UNOX system and their flow sheets are shown.

Also in the tannery waste only supply experiment, as the phosphorous content in the raw water was insufficient, phosphorous was added to conduct the experiments.

Fig.-4.4(a) Flow-Chart of Pilot Plant (Single Stage Marox System)

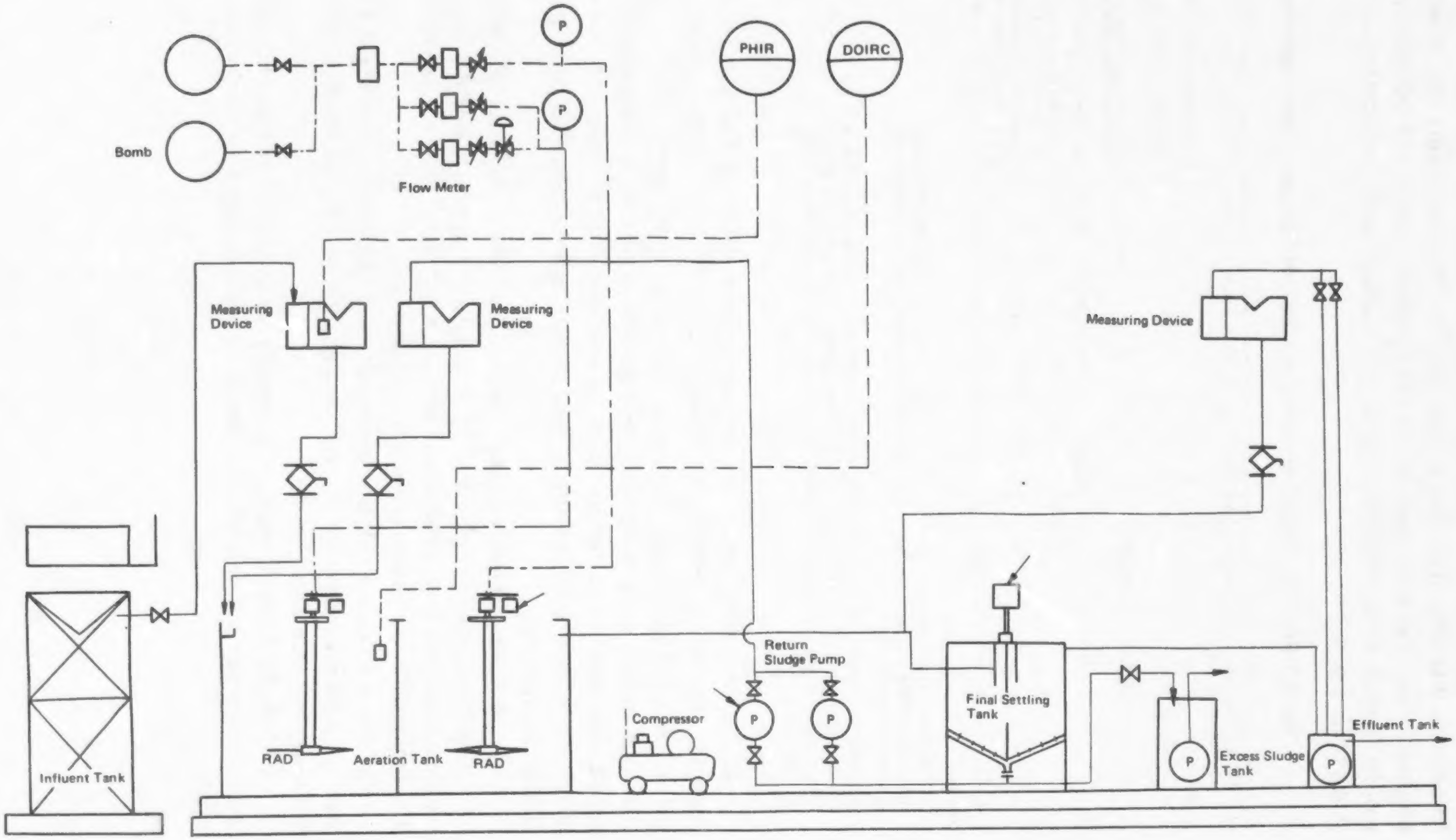


Fig.-4.4(b) Flow-Chart of Pilot Plant
(Two Stage Marox System)

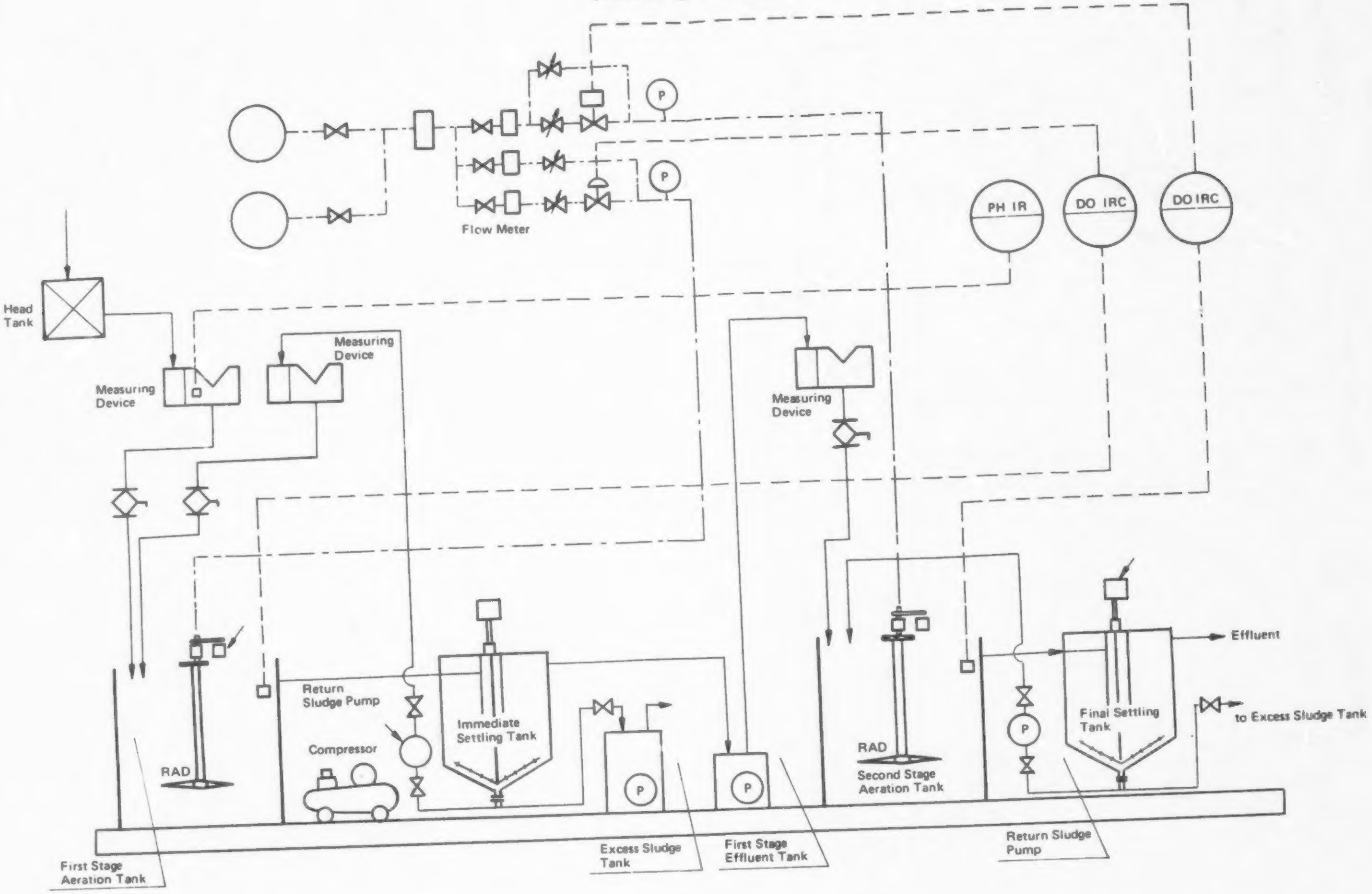


Fig.-4.5 Flow-Chart of Pilot Plant (Ebara Infilco System)

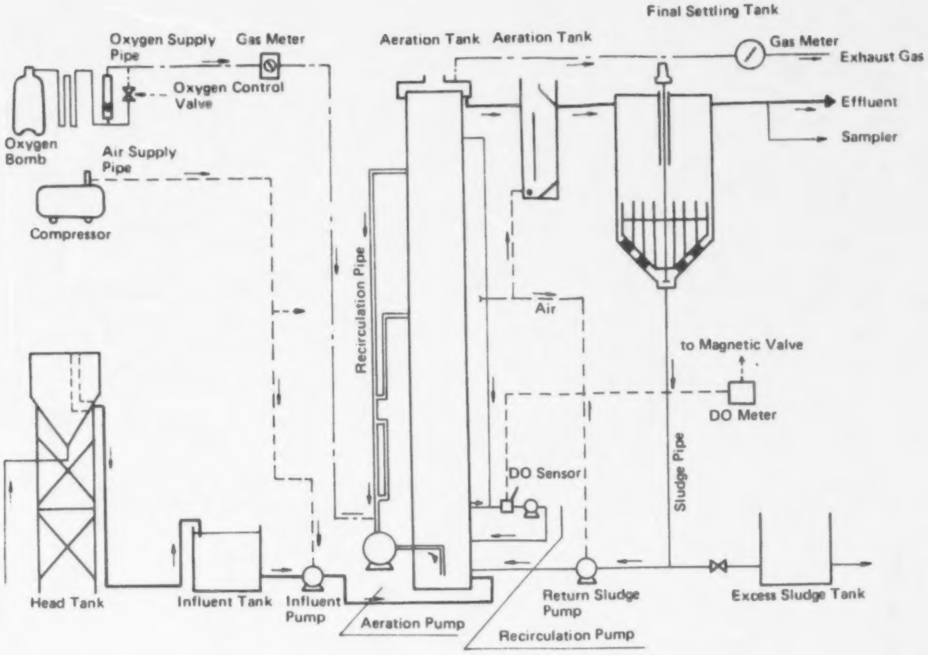


Fig.-4.6 Flow-Chart of Pilot Plant
(Unox System)

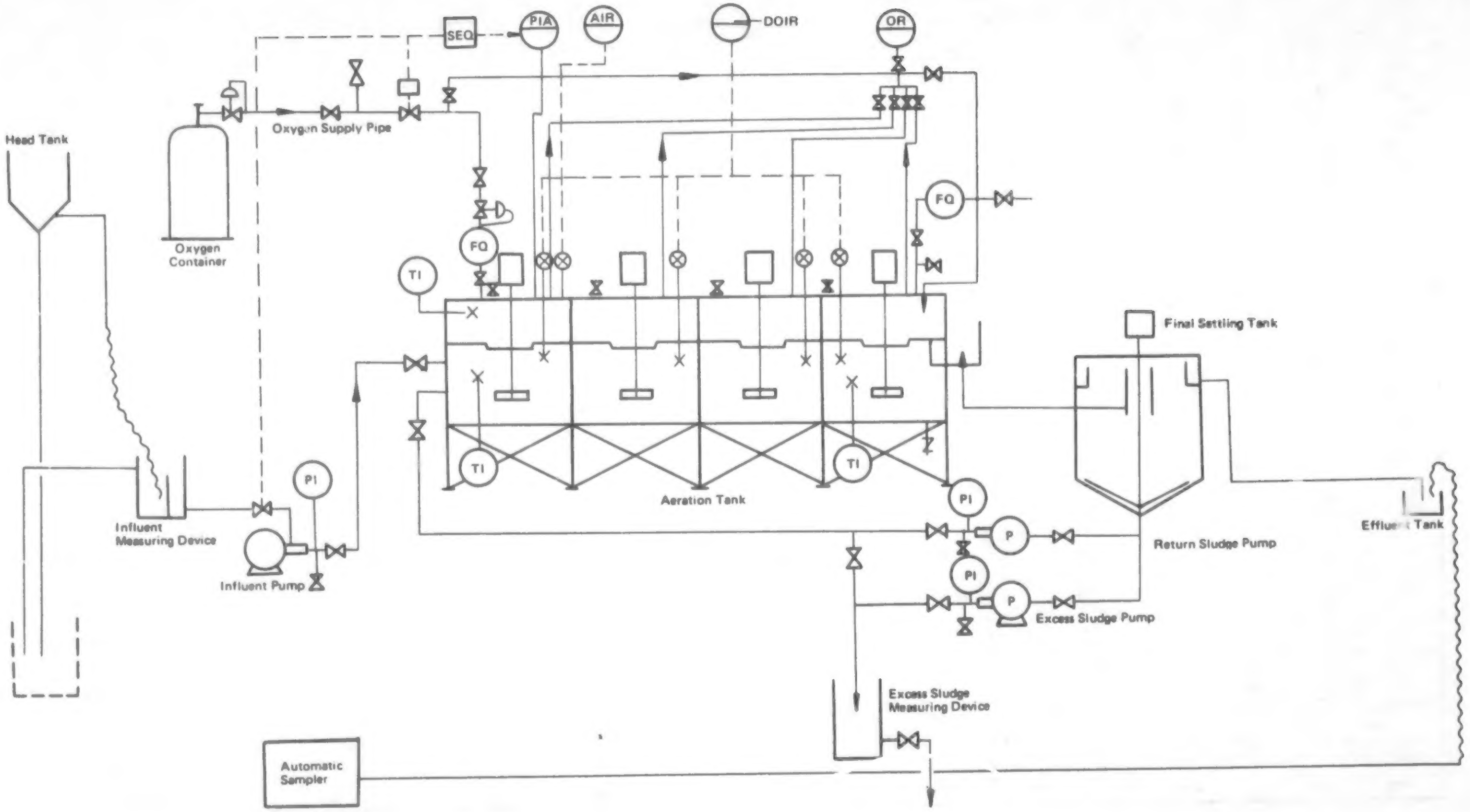


Table 5.1 Summary of Experimental Result

Name of System	Item		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
	Month of Experiment		Feb. '77	Mar.			Apr.		May	June		July		Aug.	Sept.	Oct.	Nov.		Dec.	Jan. '78		Feb.	Mar.
	Committee Decided Experimental Condition		Constant Feed of Tannery Waste Only										Varied Feed of Tannery Waste Only				Mixed Feed of Varied Tannery Waste and Constant Domestic Sewage			Varied Feed of Tannery Waste Only		Constant Feed of Tannery Waste Only	
									Evaluation						Evaluation				Evaluation		Evaluation		Evaluation
Marox System	Experiment Nos.		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
	Detention Time (hrs.)		6.0	6.0	6.0	8.0	8.0	12.0	6.0	12.0	18.0	12.0	7.6	7.6	7.6	5.7	3.8	2.8	3.9	7.6	7.6	9.0	9.0
	MLSS (g/ℓ)		13.5	10.7	8.1	8.8	7.5	10.2	7.6	6.5	8.7	8.9	5.7	5.2	4.9	3.6	3.6	3.0	3.9	4.9	5.3	7.2	6.0
	MLVSS/MLSSx100(%)		66	79	84	85	84	83	87	86	84	84	86	85	87	86	86	82	75	73	84	86	86
	Return Sludge (%)		143	155	149	257	140	222	124	86	95	73	163	148	204	394	228	171	199	248	215	195	91
	Sa·t (MLSS·T)x10³		81.0	62.4	48.6	70.4	60.0	122	45.6	78	157	107	86.6	39.5	37.2	20.5	14.1	8.4	15.2	37.2	40.3	64.8	54.0
	F/M (BOD·kg/ MLVSS·kg/d)		0.35	0.35	0.42	0.31	0.34	0.16	0.40	0.22	0.12	0.19	0.29	0.38	0.43	0.72	0.66	1.10	0.72	0.42	0.42	0.22	0.28
	SRT (days)		14.1	5.1	5.0	17.1	8.4	16.4	5.8	14.5	25.9	23.8	3.1	5.4	5.2	2.6	2.5	1.4	1.9	9.8	4.7	13.1	6.0
	BOD (mg/ℓ)	Influent	790	739	710	761	707	660	656	598	635	697	598	530	576	530	330	320	331	476	590	508	536
		Effluent	130	89.7	107	89.3	154	90.2	89.3	44.1	21.8	15.4	35.2	23.2	15.9	17.8	9.91	32.3	25.2	48.7	29.4	26.7	23.4
	COD (mg/ℓ)	Influent	434	456	405	402	416	392	373	335	350	388	354	304	336	329	215	190	194	299	377	342	408
		Effluent	136	108	115	101	114	107	96.7	80.3	57.4	45.9	66.9	55.2	50.8	42.7	31.0	42.8	39.4	63.7	59.3	60.1	53.9

Name of System	Item		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
	Stage		← Single Stage Process →									← Two Stage Process →									
Ebara Infilco System	Experiment Nos.		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
	Detention Time (hrs.)		6.2	6.2	5.9	8.8	10.1	8.5	8.5	11.7	22.9	16.5	12.0	8.1	6.2	4.7	4.1	15.8	23.2	17.5	13.6
	MLSS (g/ℓ)		13.3	11.9	9.4	9.7	9.3	9.6	11.7	9.6	10.9	10.1	9.2	9.1	7.2	6.1	5.9	8.7	9.7	10.4	11.0
	MLVSS/MLSSx100(%)		58	57	84	85	85	86	85	84	83	83	84	86	85	85	85	84	84	85	85
	Return Sludge (%)		32	42	70	41	48	51	85	44	86	67	100	114	93	82	64	66	66	71	95
	Sa·t (MLSS·T)x10³		82.5	73.8	55.5	85.4	93.9	81.6	99.5	112	250	167	110	73.7	44.6	28.7	24.2	137	225	182	150
	F/M (BOD·kg/ MLVSS·kg/d)		0.39	0.44	0.37	0.22	0.19	0.22	0.21	0.16	0.07	0.08	0.15	0.20	0.21	0.32	0.39	0.099	0.075	0.076	0.097
	SRT (days)		14.3	4.2	9.1	5.5	13.1	17.2	18.3	13.9	25.1	26.0	17.9	17.5	22.8	6.3	6.6	45.5	31.0	34.7	18.6
	BOD (mg/ℓ)	Influent	763	753	724	670	632	656	604	641	626	476	586	530	340	320	331	476	590	506	536
		Effluent	229	144	112	185	145	28.2	26.8	17.7	18.6	19.0	19.2	19.9	12.8	16.9	26.7	35.2	42.2	24.5	23.7
	COD (mg/ℓ)	Influent	458	424	400	391	413	375	345	344	370	276	338	329	224	201	194	299	377	342	408
		Effluent	183	120	99.1	116	133	74.6	61.1	54.0	49.2	50.1	48.6	42.6	40.0	32.5	36.8	51.2	63.6	59.5	63.7

Name of System	Item		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
Unox System	Experiment Nos.		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
	Detention Time (hrs.)		6.0	5.0	8.6	6.0	8.6	12.0	10.0	12.0	18.9	15.1	12.6	10.1	9.0	8.2	5.1	4.5	3.8	12.0	18.0	16.0	15.0	12.0
	MLSS (g/ℓ)		10.7	8.5	9.8	9.8	10.2	8.5	8.2	9.3	8.1	8.4	8.6	8.8	9.0	9.6	6.8	7.0	6.2	7.2	5.8	9.4	10.7	11.5
	MLVSS/MLSSx100(%)		73	84	86	87	86	83	82	82	82	82	83	85	85	85	85	86	87	85	84	84	85	87
	Return Sludge (%)		41	71	100	57	80	80	67	100	158	126	105	84	85	78	48	53	64	100	150	133	100	108
	Sa·t (MLSS·T)x10³		64.2	42.5	84.3	58.8	87.7	102	82.0	112	153	127	108	88.9	81.0	78.7	34.7	31.5	23.6	86.4	104	150	161	138
	F/M (BOD·kg/ MLVSS·kg/d)		0.40	0.50	0.24	0.28	0.22	0.18	0.22	0.18	0.12	0.15	0.13	0.19	0.16	0.19	0.28	0.28	0.39	0.18	0.14	0.11	0.06	0.11
	SRT (days)		5.1	4.5	13.6	6.3	11.7	12.8	16.3	17.5	28.4	33.8	31.2	12.6	12.5	13.2	7.5	5.8	5.4	9.7	33.3	29.5	27.1	24.7
	BOD (mg/ℓ)	Influent	772	737	710	591	687	652	628	670	639	638	500	581	582	540	340	320	331	544	408	590	497	536
		Effluent	115	98.7	83.1	96.0	71.9	25.2	22.0	17.0	13.5	15.4	9.85	13.6	12.8	17.6	12.4	8.95	24.6	30.5	23.2	24.6	13.7	24.4
	COD (mg/ℓ)	Influent	436	405	513	363	412	412	347	350	376	354	356	338	344	315	224	186	193	322	266	377	327	366
		Effluent	120	105	108	99.9	93.9	68.4	50.9	47.0	45.9	52.8	38.5	43.8	38.7	39.5	34.9	30.0	36.2	51.5	45.0	49.6	58.2	66.5
	Month of Experiment		Feb. '77	Mar.		Apr.	May	June		July			Aug.	Sept.		Oct.		Nov.		Dec.		Jan. '78	Feb.	Mar.

Fig. 5.1 Experimental Results of BOD$_5$ and COD$_{Mn}$ (Marox System)

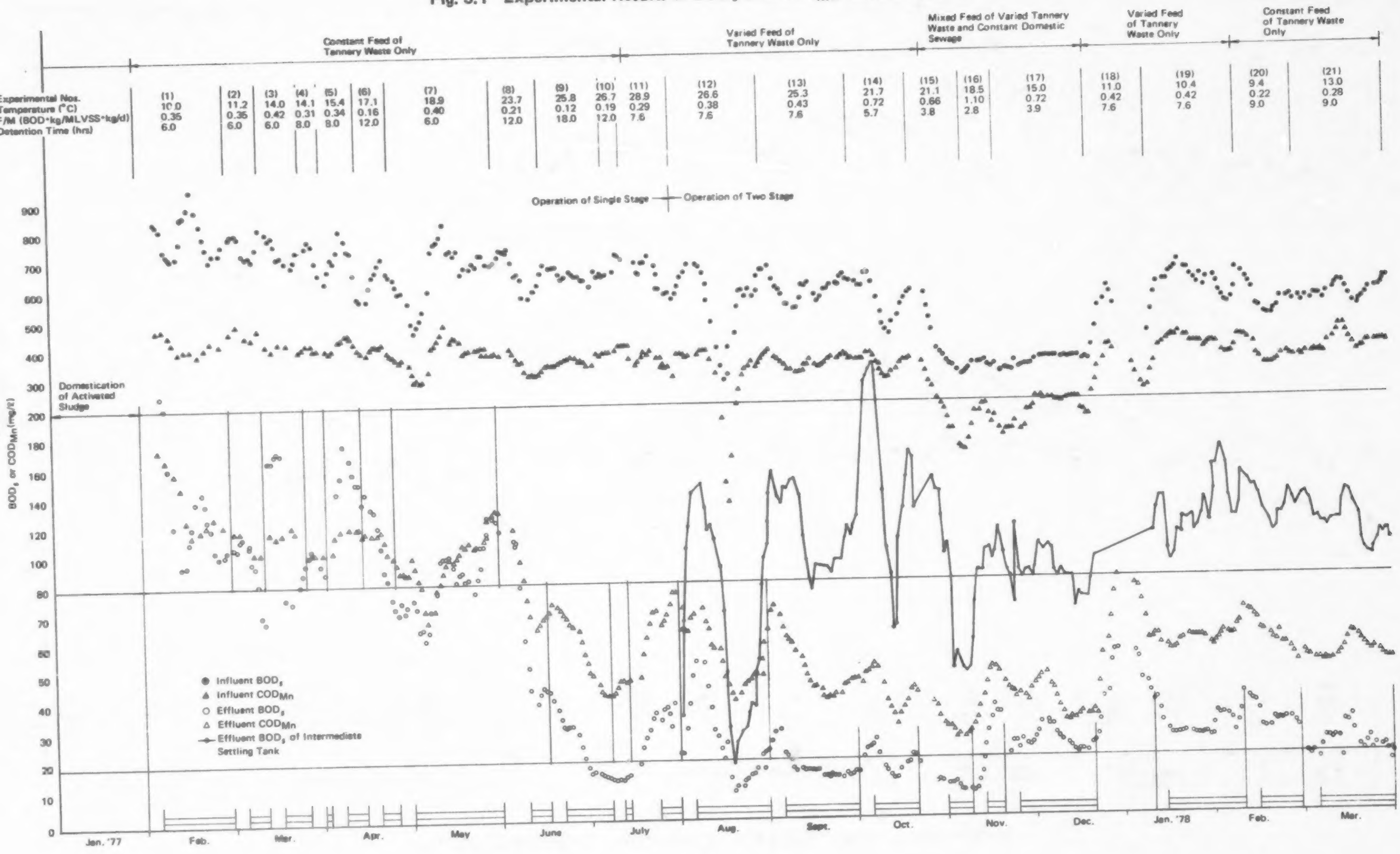


Fig. 5.2 Experimental Results of BOD_5 and COD_{Mn} (Ebara Infilco System)

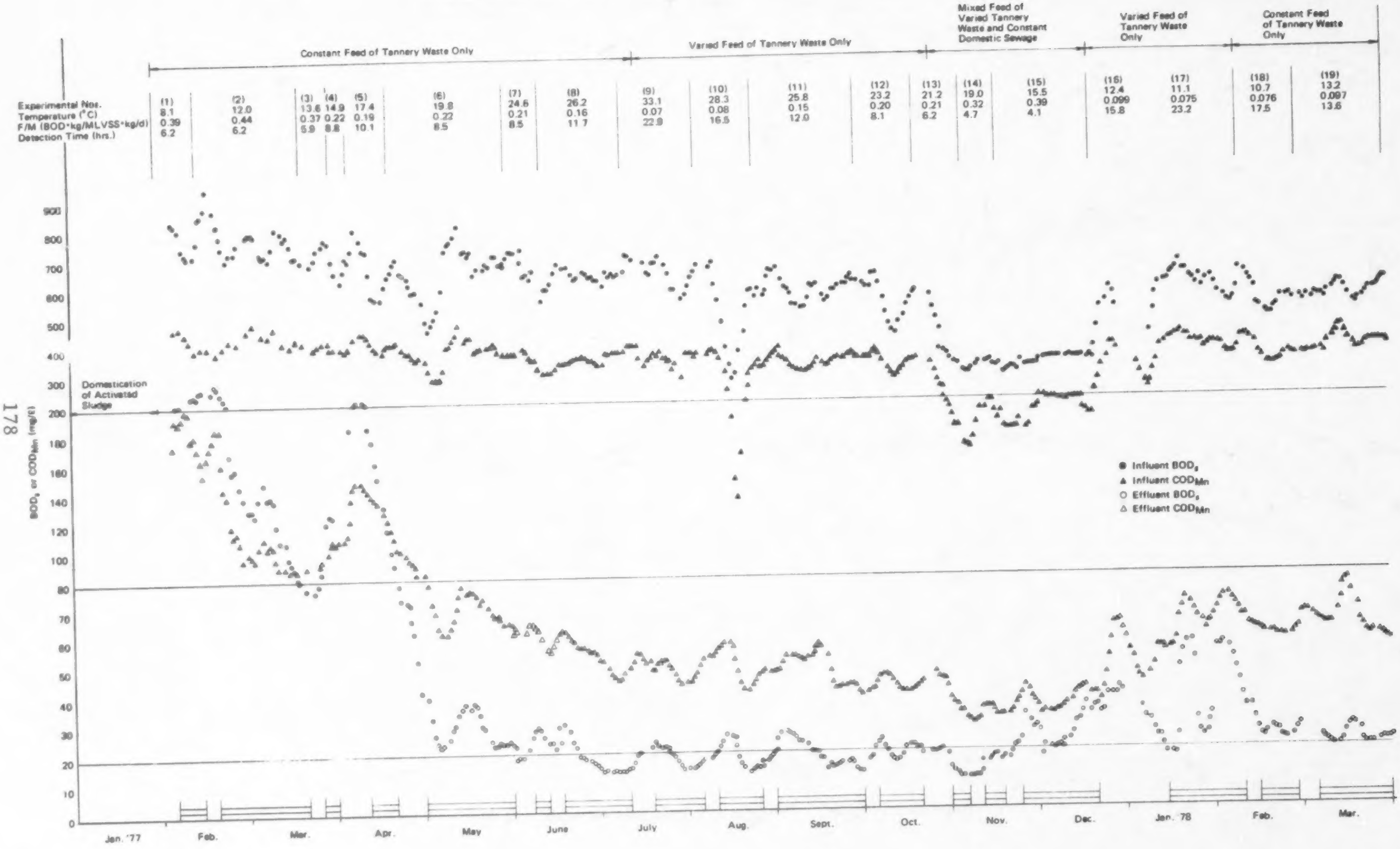
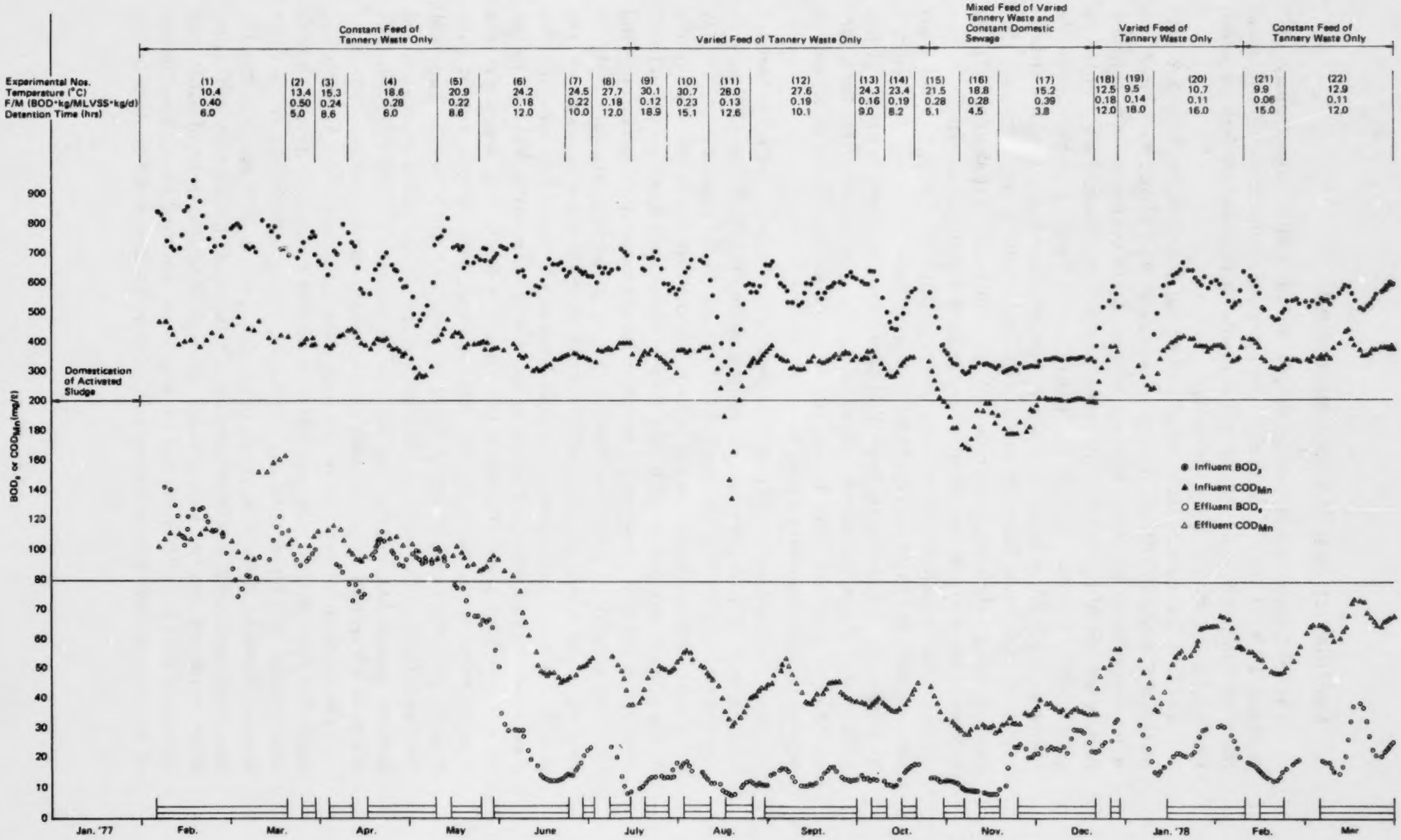


Fig. 5.3 Experimental Results of BOD_5 and COD_{Mn} (Unox System)



5. EXPERIMENT RESULTS AND DISCUSSION

The pilot plant experiments continued for 15 months from January 1977 to March 1978. Table 5.1 summarizes the operating results of the pilot plants of the three companies. And the influent and effluent BOD and COD_{Mn} are shown Fig. 5.1, Fig. 5.2 and Fig. 5.3, respectively.

The first few months after commencement of the operations have seen both BOD and COD_{Mn} in effluent frequently exceeding the 100 mg/l level due to low water temperature. However, after six months, stable operations were obtained. And by the end of December in all operating conditions, effluent quality of 15–35 mg/l of BOD and 40 – 70 mg/l of COD_{Mn} was obtained. Especially, when the operating conditions were stable, it was possible to maintain the objective effluent quality of BOD being below 20 mg/l and COD_{Mm} below 50 mg/l. However, after January, being influenced by the drop in water temperature, both BOD and COD_{Mn} in effluent began to raise again despite keeping the F/M ratio at a low level.

From the resutls of the experiment on constant feed of tannery waste water only and the experiment on varied feed of tannery waste water only, if the latter is to be operated with a slightly lower F/M ratio than the former, it was found that similar effluent can be obtained. However, to ensure stable operation for a long period of time, it was found appropriate to introduce the influent as constant as possible by providing a equalization tank.

As the experiment on joint tannery waste and domestic sewage was operated at a higher F/M ratio than that on tannery waste only, BOD and COD_{Mn} concentrations in the effluent climbed higher than those in the experiment on tannery waste water only. Especially, in the latter half of this experiment, the water quality of the effluent was influenced by the drop in water temperature of the influent. However, if the F/M ratio can be kept at a lower level in this experiment, it will obviously bring about stable performance. During this period, although the experiment on mixed feed of constant feed of tannery waste and domestic sewage was not possible due to time restrictions, it is considered that it would have brought about good results with regards stable operation and treated water quality when conducted. To maintain BOD below 20 mg/l in the effluent in the tannery wastewater only experiment, it is necessary to constantly maintain BOD removal rate at more than 97 percent, which is very difficult to sustain in many cases in biological treatment. On the other hand, in the joint treatment with tannery wastewater and domestic sewage, BOD removal rate needs to be sustained at about 95%, thus from this point it is regarded better to select the joint treatment.

The operation of MAROX system pilot plant commenced from August in two stages. But since the F/M ratio was kept at an extremely high level, its effluent BOD concentration was higher than the single-stage pilot plant effluent of the other systems. However, if the F/M ratio can be kept lower in the operation than the experiment conducted, it is considered possible to sustain the objective BOD value. When employing the two stage process, although it has the merit of conducting operations at a higher F/M ratio than the single-stage process, it has the disadvantage of requiring an intermediate settling tank. Further studies should be made before selecting either the single stage process or the two stage process.

The influent pH was between the range of 9 and 11.5, but pH of pilot plant effluent even without pH control was maintained between the range of 6.5 and 7.5. From these facts, when introducing the oxygen activated sludge process, there is no need of performing pH control. The drop in pH level in the effluent was observed more markedly in the covered type aeration tank than the open type aeration tank.

SVI of the mixed liquor was in the range of 50 – 160 and was normally kept at a level between 80 and 110. In either experimental conditions, there was no bulking of activated sludge observed. Similar values of SVI were obtained in both the experiment on tannery wastewater only and on the experiment on tannery wastewater mixed with domestic sewage.

In general, at the pilot plant experiment, both the surface area rate of the final settling tank and the settling time have shown a tendency of becoming larger than the full scale actual facility. The return sludge concentration obtained in this experiment was 15 – 30 g/l by the single stage tannery wastewater only feed process and between 11 and 16 g/l by the two-stage process. The return sludge by the experiment on mixed feed of tannery wastewater and domestic sludge was between 14 and 21 g/l by the single-stage process and 8 and 11 g/l by the two-stage process. Also MLSS concentration in the aeration tank was between 8 and 10 g/l by the experiment on tannery wastewater only feed, and between 6 and 7 g/l by the experiment on combined feed of tannery waste and domestic sewage. In the two-stage process, these was a tendency that the MLSS concentration in the aeration tank was lower than the single-stage process in both experiments. This is reasoned to the fact that in the two-stage process the return sludge ratio had to be kept at a high value of between 257 and 73%.

SRT of the pilot plant showed a marked fluctuation of between 1.4 and 34 days. But, on the average, favorable effluent quality was obtained when SRT was maintained between 10 and 15 days. Even in these instances the nitrification has only slightly occurred. The total alkalinity and ammonia nitrogen in the effluent have shown a tendency of becoming higher than in the influent.

The oxygen consumption in the aeration tank was 0.62 – 1.73 kg O_2 per removed BOD 1 kg. When performing operations to hold excess sludge in the system, this value had further increased to reach a maximum 1.91 kg. The oxygen utilization ratio in the aeration tank was between 87 and 99% although actual measurements were only made of the covered type aeration tank. In general, the value exceeded 90% showing extremely good efficiency. The use of an oxygen cylinder was quoted as the reason.

The waste activated sludge volume have shown a great fluctuation of between 0.22 and 1.34 kg per BOD 1 kg. In general, when the oxygen consumption per removed BOD increased, the sludge production have shown a tendency of decreasing. In the two-stage process, oxygen consumption per BOD removal was smaller than in the single-stage process but a tendency of the increase of sludge production have been recognized.

The accumulation of chromium in sludge is estimated to reach between 40 and 50 mg per 1 kg of dry solids when viewing the results of the experiment on tannery waste only feed. Even in the case of the experiment on the combined feed of tannery and domestic sewage, the accumulation of chromium compound is estimated to reach a maximum 40 mg per 1 kg of dry solids of sludge. Viewing from the fact that chromium compound of high concentration do accumulate in the sludge, extra care must be exerted to the disposal of such sludge.

Resulting from the accumulation of chromium in sludge, the total chromium concentration of between 6 and 20 mg/l contained in the influent have decreased to below 0.6 mg/l in the effluent, proving the fact that the oxygen activated sludge process is also effective in total chromium removal.

6. SUMMARY

The study was performed to compile the results of the oxygen activated sludge process treating tannery waste flowing into the Tobu Sewage Treatment Plant in Himeji city. However, detailed analysis of the results of the experiment are not contained in this report due to lack of ample time after completing the tests.

But it was truely fortunate that Mr. W. J. Lacy and Mr. R. S. Burd who both represented the American team at the 5th U.S.–Japan Conference on Sewage Treatment Technology have taken the trouble to extend their field trip to Himeji to inspect the experimental facilities.

The first stage of construction work of the Sewage Treatment Plant is expected to start in 1979, and the actual operation to commence during the 1982 fiscal year.

The results obtained from the experiment can be summarized as follows:

(1) It is appropriate to employ the oxygen activated sludge process for tannery waste. However, extra care must be exerted to the operation and maintenance of the process. When the F/M ratio is kept at below 0.1 kg.BOD/MLVSS. kg/d, it is possible to obtain effluent with BOD 20 mg/l and BOD removal rate 97 percent.

(2) Fluctuation of tannery waste quantity and quality is great. But if the F/M ratio can be slightly lowered at the time of constant feed, it is possible to obtain effluent with BOD of below 20 mg/l even when there is a fluctuation of influent flow. However, when considering the long period use of the biological treatment facility for stable operation and maintenance, it is desirable to provide a equalization tank for constant supply of influent.

(3) It is advisable to introduce the method of supplying tannery waste combined with domestic sewage for stable treatment. When the mixing ratio of tannery waste and domestic sewage is 1:1, even when F/M ratio is more than 0.2 kg.BOD/MLVSS.kg/d, it is possible to maintain effluent BOD below 20mg/l.

(4) When the two-stage oxygen activated sludge process is employed, although it is possible to operate with a high F/M ratio, it requires an intermediate tank. From the economical standpoint and so forth, further studies should be made before selecting either the single-stage process or the two-stage process.

(5) In the treatment of tannery waste by the oxygen activated sludge process, pH control is not necessary. Judging from the experiment results, SVI is maintained at around 100 and activated sludge is free from bulking. The removal of total chromium is effectively done, but more than 50 mg of total chrome per 1 kg of dry solids accumulate in sludge, thus extra care must be exerted in the disposal of

the sludge.

(6) The oxygen consumption in the aeration tank was between 0.62 and 1.73 kilogram O_2 per 1 kg of BOD removed. Also the sludge production was between 0.22 and 1.34 kg per 1 kg of BOD removed. In general, when the oxygen consumption per BOD removed increased, the sludge production have shown a tendency of decreasing.

CHAPTER 6

SERVERAL APPLICATIONS OF PURE OXYGEN ACTIVATED SLUDGE SYSTEMS FOR TREATING MIXTURE OF HIGH ORGANIC INDUSTRIAL AND DOMESTIC WASTEWATERS

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1. OVER VIEW OF PURE OXYGEN ACTIVATED SLUDGE PROCESS IN JAPAN

When compared with the United States, the number of examples of applying pure oxygen activated sludge process to publicly owned sewage plants is small in this country (Table-1). This may be reasoned to the fact that people no more think it necessary to change the air activated sludge system to another method due to the completion of designing techniques and the rich experiences in management and operation. Only when waste water can not be treated effectively by the air system, then the oxygen system is basically used. Until mid 1977, the cases of publicly owned pure oxygen activated sludge process had been only 4.

In Octover, 1977, the nation's largest UNOX System with a capacity of 40,000 m^3/day was completed and immediately put in operation. This is the first unit that was completed by the national construction grant.

The open type activated sludge process is now available in Japan thanks to the good offices of Ataka Construction Company, K.K. This type is applied at two industrial waste water treatment plants in 1978 for pretreatment purposes.

With the aim of compiling a manual for the pure oxygen activated sludge process, we have decided to gather data and the features of this process. Hitherto, the pure oxygen activated sludge process was considered highly economical only when the BOD concentration contained in the raw water to be treated was high. Also, when treating municipal sewage, its power cost was considered higher than that of the air system. To confirm the reliability of this general idea is also one of our aims in this survey.

We have conducted performance tests for both the UNOX system and the MAROX system by full plant test and pilot plant test respectively based on the available data.

Table-1-a Unox Plants in Japan in Operation as of August 1978

No.	Location		Application	Aeration Tank Design			Oxygen Supply		Aerator	On Stream
				Flow (m^3/day)	Retention Time ((TO) hrs)	BOD, Inlet mg/l	Capacity (tons/day)	System		
1.	Kasuga S.T.P.	Oita City, Oita Pref.	Municipal Raw Degritted	1,000	1.8	200	0.4	PSA	SA	Oct., 1972
2.	Ikuta S.T.P.	Kawasaki City Kanagawa Pref.	Municipal 1 ry Effluent	2,300	1.57	140	0.6	PSA	SAB	June, 1973
3.	Uenodai S.T.P. Japan Housing Corp.	Kamifukuoka City Saitama Pref.	Municipal Raw	1,950	1.70	200	0.6	PSA	SAB	Dec., 1973
4.	Gotsu S.T.P.	Katano City Osaka	Municipal Degritted	2,750	2.0	274	1.1	PSA	SA	May, 1974
5.	Kisshoin S.T.P.	Kyoto City Kyoto	Municipal with Dye Wastes 1 ry Effluent	40.000	3.0	220	15.0	PSA	SAB	Sept., 1977
6.	Nissho Kayaku Co., Ltd.	Oita City, Oita Pref.	Petrochemical Waste	2.800	10.0	1,700	5.0	PSA	SA	Sept., 1972
7.	Showa Neoprene K.K.	Kawasaki City Kanagawa Pref.	Synthetic Rubber Waste	3,000	3.6	190	1.0	Pipe	SAB	Oct., 1972
8.	Sankyo Kokusaku Pulp Co., Ltd.	Iwakuni City Yamaguchi Pref.	Pulp & Paper Waste	3.360	9.0	2,000	8.3	PSA	SA	July, 1973
9.	Jujo Paper Co., Ltd.	Kushiro City Hokkaido	Pulp & Paper Waste	6.000	8.0	2,000	18.4	PSA	SAB	Sept., 1973
10.	Oji Paper Co., Ltd.	Kasugai City Aichi Pref.	Pulp & Paper Waste	70.000	2.4	286	25.0	PSA	SA	July, 1974
11.	Oji Paper Co., Ltd. (Expansion)	Kasugai City Aichi Pref.	Pulp & Paper Waste	22.000	2.6	380	16.0	PSA	SA	April, 1978
12.	Oji Paper Co., Ltd.	Tomakomai City Hokkaido	Pulp & Paper	50.000	3.7	600	27	Pipe	SA	Aug., 1974
13.	Mitsubishi Chemical Industries Ltd.	Kurashiki City Okayama Pref.	Petrochemical Waste	7,200	9.3	970	8.5	Pipe	SA	Feb., 1975

Table-1-b Unox Plants in Japan in Operation as of August 1978

No.	Location		Application	Aeration Tank Design			Oxygen Supply		Aerator	On Stream
				Flow (m^3/day)	Retention Time ((TO) Hrs)	BOD, Inlet (mg/l)	Capacity (tons/day)	System		
14.	Sumitomo Chemical Co., Ltd.	Ichihara City Chiba Pref.	Petrochemical Waste	3,000	9.1	1,320	3.7	Pipe	SAB	June, 1975
15.	Electro Chemical Industrial Co., Ltd.	Ichihara City Chiba Pref.	Petrochemical Waste	10,000	6.6	730	9.2	Pipe	SA	July, 1975
16.	Tokiwa Sangyo K.K.	Oware Asahi City Aichi Pref.	Pulp & Paper Waste	14,000	1.8	150	2.4	PSA	SAB	Oct., 1975
17.	Mitsubishi Chemical Industries Ltd.	Kitakyushu City Fukuoka Pref.	Dyestuff Waste	11,700	8.15	830	11.1	Pipe	SAB	Jan., 1976
18.	Mitsui Toatsu Chemicals, Inc.	Takaishi City Osaka	Petrochemical	6,480	5.5	1,100	7.8	Pipe	SA	Feb., 1976
19.	Sapporo Breweries, Ltd.	Sapporo City Hokkaido	Brewery Waste	3,000	8.0	1,330	5.0	PSA	SA	April, 1976
20.	Nippon Kasei Chemical Co., Ltd.	Iwaki City Fukushima Pref.	Coke Oven Waste	3,900	5.6	620	2.85	Pipe	SA	May, 1976
21.	Koa Kogyo K.K.	Fuji City Shizuoka Pref.	Pulp & Paper Waste	40,000	2.0	220	33.0	PSA	SAB	June, 1976
22.	Toyo Pulp Co., Ltd	Kure City Hiroshima Pref.	Pulp & Paper Waste	11,000	3.6	540	7.2	Liquid	SAB	Sept., 1976
23.	Otake Shigyo K.K.	Otake City Hiroshima Pref.	Pulp & Paper Waste	33,000	2.0	190	11.4	PSA	SAB	Oct., 1976
24.	Sumitomo Chemical Co., Ltd.	Niihama City Ehime Pref.	Chemical Waste Waste	180	4.0	300 (BOD$_{5}$)	0.06	Pipe	SAB	April, 1977

Table-1-c Unox Plants in Japan in Construction in Japan as of August 1978

No.	Location		Application	Aeration Tank Design			Oxygen Supply		Aerator	On Stream
				Flow (m^3/day)	Retention Time ((TO) Hrs)	BOD, Inlet (mg/l)	Capacity (tons/day)	System		
1.	Todoroki S.T.P. (Phase I)	Kawasaki City Kanagawa Pref.	Municipal 1 ry Effluent	60,000	1.93	105	15	PSA	SAB	1981
2.	Ishinomaki-Tobu S.T.P. (Phase I)	Ishinomaki City Miyagi Pref.	Municipal with Fish Processing Waste	10,000	≥ 5.0	700	11	PSA	SAB	1981

Note: S.T.P. = Sewage Treatment Plant
PSA = On-site pressure swing adsorption exygen gas generator
Liquid = On-site liquid oxygen storage and vaporization
Pipe = Pipeline transport of oxygen gas from a off-site oxygen generating facility
SA = Surface aerators
SAB = Surface aerators with bottom mixers

2. COVERED OXYGEN SYSTEM FOR HIGH BOD MUNICIPAL WASTE WATER TREATMENT

– Case of Kyoto City –

2.1 BOD REMOVAL

In this sewage treatment plant are two trunk sewers, A and B, through which raw sewage flows in. The A chiefly deals with industrial waste water while the B handles domestic sewage. The former contains an average 250 mg/l of BOD while the latter 120 mg/l. By varying the mixing rate of both wastewaters, the BOD concentration in the raw sewage can be altered. Under the several different influent BOD concentrations, UNOX's performance was observed. BOD removal, SVI, and oxygen consumption were our main concerns. Table 2 shows the summary of results of the test.

Even in this case when the BOD load is as large as 2 kg/m^3/day, the effluent BOD did not exceed 20 mg/l. SVI registered a low value of 100. And the BOD·SS load at this time (BOD/kg·SS/day) was 0.38.

At this plant, the air system is also employed in parallel. In the air system, when the BOD load exceeded 1 kg/m^3/day, bulking generated.

Table-2 Performance of Unox in Kyoto

Test Item		I	II	III
Percentage of Industrial Waste-Water (%)		50	100	100
Temperature in Reactor (°C)		17.9	19.9	19.9
Aeration Time	Q (hr)	2.8	3.2	3.0
	(Q+R) (hr)	2.1	2.1	2.0
BOD Load in Tank (kg/m^3/day)		1.36	1.74	2.01
BOD SS Load (kg/m^3/day)		0.31	0.33	0.38
MLSS (mg/l)		4.552	5.308	5.383
MLVSS/MLSS (%)		80.4	74.8	76.7
Concentration of Return Sludge (mg/l)		14,976	12,697	11,538
SVI		191	117	116
Final Clarifier (For Q Base) (hr)		3.5	3.9	3.6
Final Clarifier (Overflow Rate) (m^3/m^3 day)		24.3	21.4	23.3
BOD	(Influent) (mg/l)	163	233	247
	(Effluent) (mg/l)	14	18	18
COD	(Influent) (mg/l)	103	173	180
	(Effluent) (mg/l)	35	70	74
SS	(Influent) (mg/l)	74	120	136
	(Effluent) (mg/l)	11	22	25
Oxygen	BOD_R/MLVSS (kg/kg)	0.3, 0.4, 0.5	0.3, 0.4, 0.5	—
	Used O_2/BOD_R (kg/kg)	1.05,0.86.1.14	1.07,1.36,1.17	
	(Efficiency) (%)	95.5	87.3	—

2.2 NUISANCE FREE DESIGN

As this plant was constructed in the center of a residential area, extra care was exerted to finish the facade in plesant apperance in design. The aeration tank and PSA were totally housed in the structures. Also, to save space of the building lot, the aeration tank was constructed atop the final Clarifier.

All sources of noise including PSA were housed, thus the noise level of the sewage treatment plant at the border line was within the standard values.

Odorants contained in the exhaust gas of both the pure oxygen activated sludge process and the air system were checked. The Offensive Odor Control Law stipulates

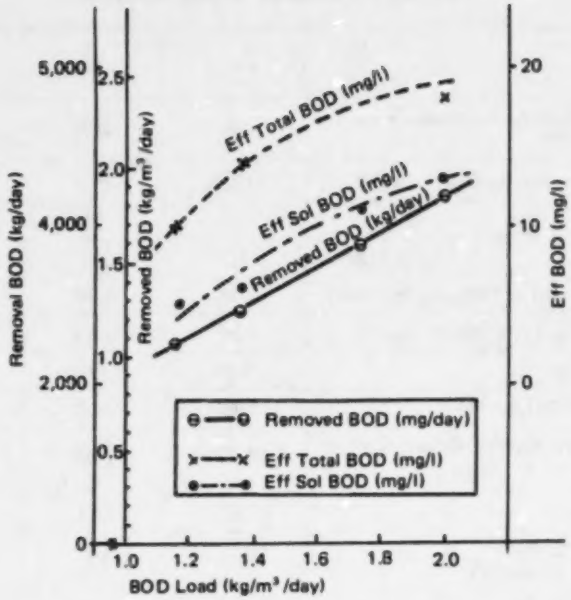
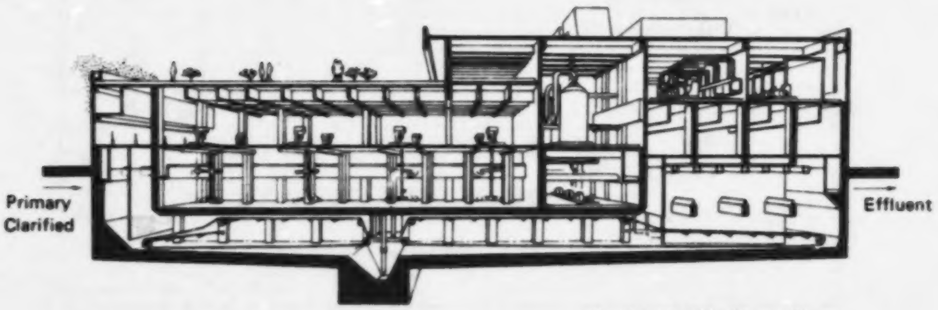


Fig.-1 BOD Load vs Eff BOD and Removed BOD Weight



1. Aeration Machinery Room
2. Aeration Tank
3. Final Clarifier
4. Return Sludge Pump
5. Electric Room
6. Oxygen Generating Unit (PSA)
7. Air Compressor Room
8. Ventilation Rook

Fig.-2 Covered Aeration Tank and

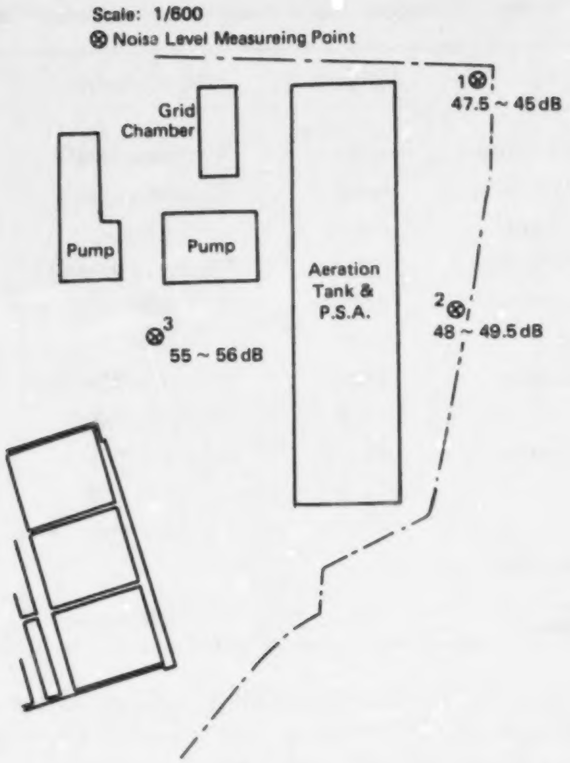


Fig.-3 Noise Level

the permissible level of the odor threshold concentration as shown in Table 3.

Table-3 Allowable Maximum Concentration of Ill Smelling Material

Odor Density / Odorant	Odorant Concentration		
	2.5	3.0	3.5
Ammonia	1.0 ppm	2.0 ppm	5.0 ppm
Hydrogen Sulfide	0.02	0.06	0.2
Methyl Sulfide	0.01	0.04	0.2
Methyl Disulfide	0.009	0.003	0.1
Trimethylamine	0.005	0.002	0.07
Acetaldehyde	0.05	0.1	0.5
Styrene	0.4	0.8	2.0

The measuring results of odorants contained in the exhaust gas of the pure oxygen activated sludge process and the air system are shown in Table 4.

Table-4 Ill Smelling Material Concentration at Aeration Tank

	Air System	Oxygen System	Allowable Limit
	(ppm)	(ppm)	
Hydrogen Sulfide	0.002	ND (Below 0.003)	
Methyl Mercaptan	0.003	ND (Below 0.001)	0.002
Methyl Sulfide	0.002	0.003	0.01
Dimethyl Sulfide	0.0003	ND (Below 0.0001)	0.009
Acetaldehydle	0.002	0.003	0.05
Ammonia	0.029	0.017	1.0
Trimethylamine	0.0003	ND (Below 0.0002)	0.005
Styrene	0.005	ND (Below 0.005)	0.4
Carbon Dioxide	0.9 (%)	5.5 (%)	
Oxygen	19.4 (%)	35.0 (%)	
Nitrogen	79.7 (%)	59.5 (%)	
Carbonyl Sulfide	–	–	
(Formaline) Formaldehyde	0.24 ppm	0.008 ppm	

The concentrations of odorants contained in the exhaust gas of both systems were below the regulated standard values. And in general the odorant concentration values of the pure oxygen activated sludge process were more than those of the air system.

Table 5 expresses the emission volume of odorants in air per unit sewage volume. This Table shows that when comparing the emission volume of odorants in the air by the pure oxygen activated sludge process and the air system, the emission volume by the pure oxygen activated sludge method is smaller from 1/1000th to 1/100th.

Table-5 Volume of Smelling Material in a Unit Volume of Sewage

	Oxygen	Air	Unit
Ammonia	0.191	170	10^{-9} m^3/m^3
Methyl Mercaptan	Below 0.011	2.76	
Hydrogen Sulfide	ND	15.3	
Methyl Sulfide	0.034	291	
Demethyl Sulfide	ND	2.76	
Trimethylamine	0.034	20.3	
Styrene	ND	Below 646	
Oxygen	0.004	1.46	m^3/m^3
Carbondioxide	0.0006	0.055	
Nitrogen, Argon, etc.	0.0067	5.86	

3. COVERED OXYGEN SYSTEM FOR DOMESTIC WASTE WATER TREATMENT

– CASES OF 4 PLANTS: Kawasaki City, Katano City, Oita City and Uenodai –

3.1 BOD REMOVAL

The subject four cities have a long experience in treating municipal sewage with the pure oxygen activated sludge system. Table 6 shows the performance data obtained at the four cities.

Table-6 Performance Data of Oxygen System Treating Domestic Wastewater

	Kawasaki	Katano	Oita	Uenodai
Capacity m^3/day	2,300	2,750	1,300	1,950
Aeration Time for Q (hr)	1.6	2	1.8	1.7
Primary Clarifier	Yes	No	No	No
Raw BOD (mg/l)	130 ~ 360	130 ~ 200	90 ~ 180	20 ~ 240
Eff BOD (mg/l)	7 ~ 18	7 ~ 16	9 ~ 20	5 ~ 20
SVI	150	100	150	100
Number of Operatior	11	8	5	1

3.2 COMMENTS BY EACH CITY WITH REGARDS OXYGEN SYSTEM

At Kawasaki-city, the hitherto air system had a slight tendency of causing bulking of sludge. Therefore, the total performance was instable. However, after employing the oxygen system, bulking has never occurred. Also, the oxygen system seems to be flexible against shock loads.

At Katano-city, both the oxygen system and the air system are put into operation in parallel at the same treatment plant. The performance of the oxygen system is considered much more stable than the air system. But, the concentration of return sludge in the oxygen system do not reach the designated value of mg/l, owing to the lack of ability of the rectangular clarifier to condense sludge. There should be some improvement made with regards the shape of the basin.

As the oxygen system at Uenodai is fully automatic, it is possible to conduct the central control and monitoring system. That is why only one operator can mange the plant. Despite the plant being located close to a school and a cluster of houses, there is no complaint brought against the plant with regards ill smell due to the smallness of exhaust gas volume.

4. POWER COST

Fig. 4 compares the power costs of the UNOX system and the air system. Both are the yearly average costs and show no remarkable differences.

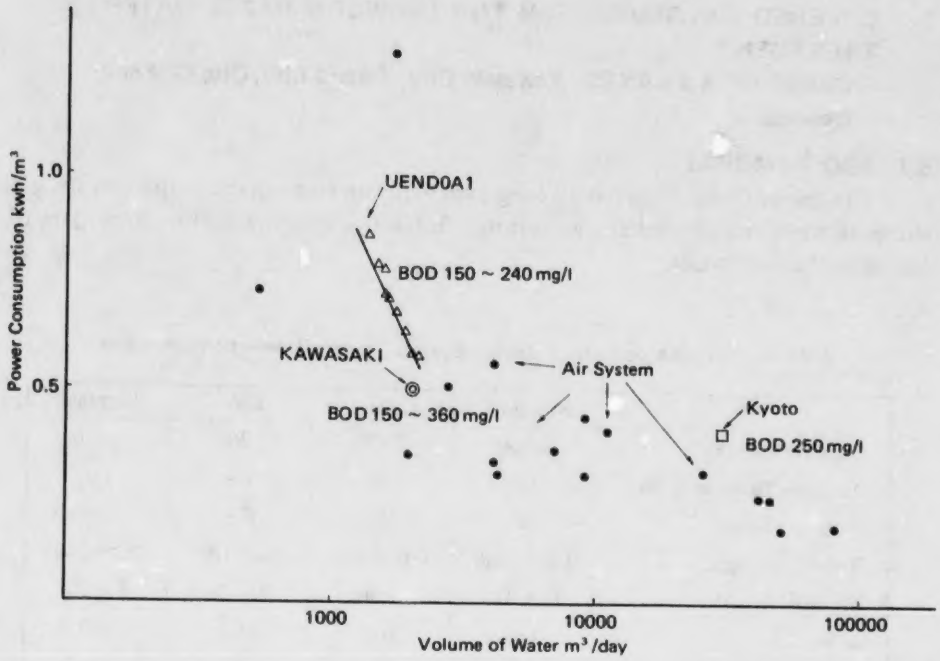


Fig.-4 Comparasion of Power Costs of the Oxygen System and the Air System

5. OPEN TANK PURE OXYGEN ACTIVATED SLUDGE PROCESS

– Case of Pilot Plants –

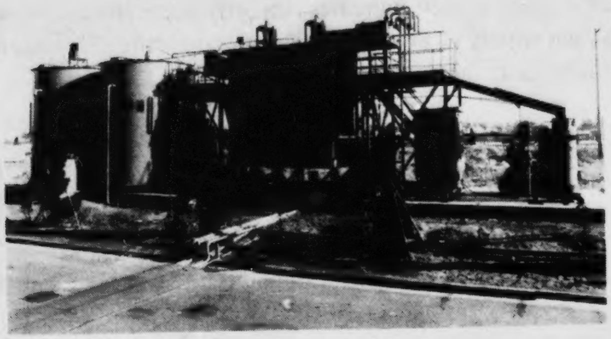
The function of the open tank pure oxygen activated sludge process was examined at a pilot plant in three places. Photo 1 shows the total view of the pilot plant. The plant dimensions are: Depth 2.5 meters, width 1.5 meters, and length 3 meters. The effective capacity is 12 m³, the RAD diameter 50 cm (diffusion area 0.0214 m²), rotating speed 400 rpm, and the oxygen supplying capacity 12.2 kg/day. The greatest interest in the open tank oxygen activated utility sludge system was to find out the oxygen utility efficiency. The oxygen utility efficiency can be obtained from the following equation:

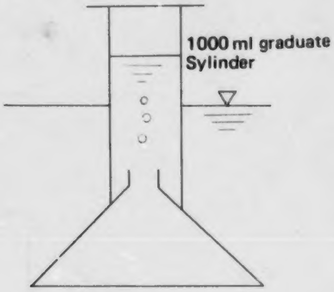
$$\left\{ \text{Used } O_2 + \text{effluent } O_2 - \text{influent } O_2 \right\} / \text{supplied } O_2$$

The amount of used oxygen can be obtained by substracting the oxygen contained in the exhaust gas from the supplied oxygen.

- The collection and measurement of exhaust has were conducted as follow:

As shown on the left, a 30 mm funnel and a 1000 ml graduate cylinder filled with water were used to collect the exhaust gas that ascended and released in the air. The exhaust gas volume was measured by measuring the volume of the exhaust gas collected in the graduate cylinder and computing the elapsed time.





The measurement results are show in Table 7.

Table-7 Data for O_2 Material Balance in Marox Pilot Plant

O_2 Supply (Nm^3/day)	O_2 Sup/ std (%)	Off Gas (Nm^3/day)	Off Gas/O_2 Supply (%)	Composition of Off Gas			Efficien- cy (%)	Place (RAD)
				O_2	CO_2	Other		
				(%)				
(1) 16.1	175	4.03	25.0	76	5	19	81	No. 1
(2) 16.1	175	3.86	24.0	76	7	15	81	No. 2
(3) 40.8	443	12.2	30.0	79	4	17	76	No. 1
(4) 40.8	443	12.2	30.0	78	3	18	77	No. 1
(5) 6.75	73	0.464	6.87	35.7	11.0	53.3	98	No. 2
(6) 7.67	83	0.518	6.75	49.6	15.0	35.4	97	No. 2
(7) 11.9	129	0.491	4.13	39.5	12.0	48.5	98	No. 2
(8) 10.3	112	1.20	11.6	43.0	22.5	34.5	95	No. 1
(9) 14.8	161	2.05	13.9	44.4	23.1	32.5	94	No. 2

Note: The MLDO at measuring time:

(1) – (4) Both No. 1 and No. 2 were 2 – 3 mg/l
(5) – (9) Both No. 1 and No. 2 were 3 – 4 mg/l

In Fig. 5, the oxygen consumption efficiency is expressed on the axis of left ordinates and the exhaust has rate is expressed on the axis of right ordinates. With the volume of oxygen supply increases, the efficiency drops. When the designated amount of oxygen supply is given, the efficiency becomes 95 percent.

This value exceeds those of the UNOX method.

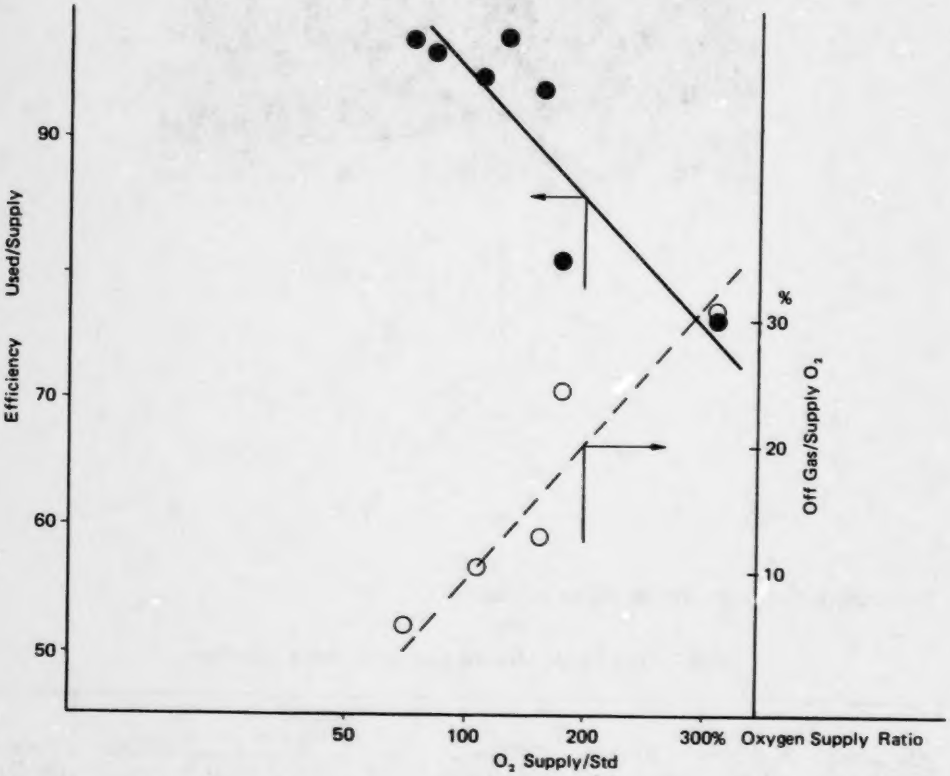


Fig.-5 Oxygen Utilization vs O_2 Supply in Open Tank Oxygen System

In Fig. 6, the actual measurement values of the relationship between BOD·MIVSS Load and required oxygen to remove unit BOD given. For comparasion, the values obtained at ENGLEWOOD and the MAROX system is DENVER were were also shown on the Figure. Our measurement values also showed similar rusults as those in the United States.

The BOD removing performance by the open tank pure oxygen activated sludge system have shown similar results with those of the covered oxygen system with no marked difference.

CHAPTER 7

REMOVAL OF COLOR FROM MUNICIPAL SEWAGE CONTAINING TEXTILE WASTEWATER

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CHAPTER 7

REMOVAL OF COLOR FROM MUNICIPAL SEWAGE CONTAINING TEXTILE WASTEWATER

1. INTRODUCTION

The cleaness or dirtiness of water should be, as a rule, determined by the amount of specific pollutants contained in the water. However, when a man judges the cleaness of a river, he is often influenced sensibly by elements which can not be specified, such as color, smell and foam of the water. Especially with regards color, there is essentially no direct or indirect regulations in the form of stream, effluent or pretreatment standards. However, even in the field of sewage treatment, there are several cases where receiving bodies of water are being offended in appearance due to the colored effluent from treatment plants which treat municipal sewage containing textile wastewater and other colored industrial wastes. This situation makes the study of color removing processes one of the major research projects. At the present stage, the method to measure the degree of color itself is insufficient. There remain many economical and technical problems to be solved including the comparison of the system of treating colored wastewater after receiving into public sewerage system with that of treating it by pretreatment facilities.

In this study, methods of color measurement are discussed and color removal from municipal sewage containing textile wastewater by activated sludge process and physical-chemical processes as tertiary application are investigated.

2. COLOR MEASUREMENT

The methods of measuring color of water can be divided into two categories: the spectrophotometric method and the visual comparison method. The former includes the mono-chromatic specification method, and the method to express the degree of color by the color difference from pure water by improving the mono-chromatic specification. The latter, however, is varied: a method of expressing degree of color by making comparison with the color of platinum – cobalt solution; a method to obtain chroma and luminance based on JIS standard color plates and to express the extent of color based on these parameters, which is stipulated in the Anti-Pollution Ordinance of Kawasaki City; and a method of expressing the degree of color by making comparisons with other suitable color scales. Furthermore, there are the sensual methods of expressing the degree of color based on the impression gained by viewing the color of water.

Generally, from the standpoint of pertinently expressing the various colors resulting from industrial wastewater, the conditions that the method of color measurement should satisfy can be listed as follows:

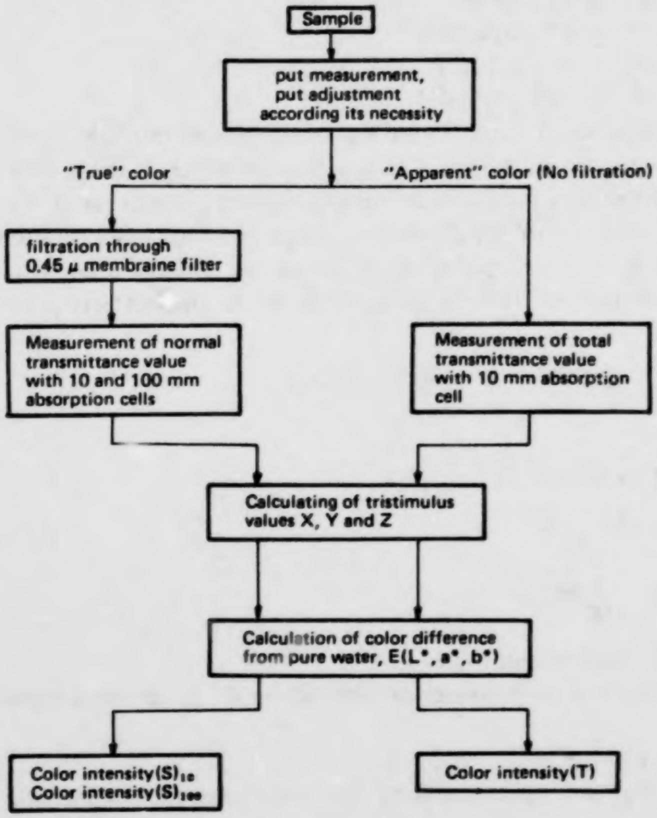
(a) Results coincide at least roughly with the visual impression of man.
(b) Reasonable comparisons are possible with regards colors of different hues.
(c) Colors by dissolved substances and by suspended matters are equally evaluated.
(d) Results are qualitative and have a good correlation, preferably linear relationship, with the concentrations of substances causing color, and

(e) The measuring method is simple and has a good reproducibility, and personal error is small.

The above-mentioned conditions are not always perfect, even containing conditions that would partially contradict with each other. However, tentatively setting these conditions as the criteria for judgement and by comparing the various measuring methods aforementioned, the color difference expression by the spectrophotometric analysis and the method described in the ordinance of Kawasaki City are considered to be effective. Accordingly, by employing the two methods, measurements of color of various dye solutions and river waters were conducted to make preliminary studies for evaluating them. The results have shown that the two methods were not fully satisfactory but applicable for practical use. However, the spectrophotometric method was superior in quantitativeness and measurements at higher concentration, thus, the method based on the color difference was to be employed in this study.

By slightly modifying the original method, the procedure for color measurement was determined as in Fig. 1. The color difference from pure water calculated by this method is referred to as 'color intensity' in this paper. The modified points are: the stricter filteration condition and the emplyment of the newer L*a*b* system (1975 – UCS1) for calculation of color difference.

Fig.-1 Procedure for Color Intensity Measurement


Sample
put measurement, put adjustment according its necessity
"True" color
"Apparent" color (No filtration)
filtration through 0.45 μ membraine filter
Measurement of normal transmittance value with 10 and 100 mm absorption cells
Measurement of total transmittance value with 10 mm absorption cell
Calculating of tristimulus values X, Y and Z
Calculation of color difference from pure water, E(L*, a*, b*)
Color intensity(S)10
Color intensity(S)100
Color intensity(T)

Since the color of water may differ greatly depending on pH of the sample, it is adjusted to be within the range of stream standard (6.5 – 8.5) by adding 10 N sodium hydroxide solutions, when pH of the sample is outside of the range. For the measurement of color by soluble substances, the normal transmittance value of the sample within the wavelengths of 400 – 700 nm is measured after 0.45 μ membrane filtration. With regards color by suspended matters, the total transmittance value of the non-filtered sample is measured within same wavelengths. The total transmittance value mentioned here is the transmittance value combining both the transmitted light and the scattered light measured by the spectrophotometer having a photo cell with integrating sphere. The measurement of the transmittance value is based on JIS Z8722 (Methods of Measurement for Color of Materials Based on the CIE 1931 Standard Colorimetric System) and is conducted at 10mm intervals by setting the transmittance value of distilled water as 100 percent, or is performed by 10 or 30 ordinate systems in spectrophotometric color measurement to calculate the tristimulus values X, Y and Z. From these values, the color difference from pure water E (L*a*b*) can be obtained from the following equation:

$$\Delta E(L^*a^*b^*) = (\Delta L^*)^2 + (\Delta a^*)^2 \quad \ldots\ldots \quad (1)$$

Here, ΔL^*, Δa^*, and Δb^* are the difference between the sample and the pure water with regards luminance parameter L^* and chromaticness parameter a^* and b^* which are defined as follows:

$$L^* = 25(100Y/Yo)^{1/3} - 16$$
$$a^* = 500\ (X/Xo)^{1/3} - (Y/Yo)^{1/3}$$
$$b^* = 200\ (Y/Yo)^{1/3} - (Z/Zo)^{1/3}$$

In which, Xo, Yo and Zo are the tristimulus of the light source. By using any spectrophotometer with Type C Light Source, which is designated by JIS Z8720 (Standard Illuminants and Sources for Colorimetry), can be set at 100 (percent).

The influence of dissolved substances and suspended matters to the total transmittance value is reported to be as follows: Let the total transmittance value be T, absorption coefficient of solution be K and scattering coefficient be S, respectively,

$$\log\left(\frac{1}{T}\right) \log\ pe^{qd} - (p-1)e^{-pd} \quad \ldots\ldots \quad (2)$$

where,

$$q = \sqrt{K(K+2S)}$$
$$p = \frac{(1+a)^2}{4a}$$
$$a = \sqrt{\frac{K}{K+2S}}$$

d: thickness of solution.

When there is no influence of turbidity or K S, the above equation is reduced to:

$$\log(1/T) \propto Kd \quad \ldots\ldots \quad (3)$$

log (1/T) is in proportion to the concentration. On the other hand, when considering the case when $K \fallingdotseq 0$, equation (2) becomes,

$$1/T \propto Sd \qquad \text{(4)}$$

(1/T) is in proportion to the concentration. Accordingly, color by dissolved and suspended substances do not exhibit the same weight. However, while (1/T) is small, log (1/T) is nearly equal to (1/T), and the combined influence of the two can be expressed in a single parameter by incorporating the total transmittance values.

The light paths for measurement of the total transmittance values are 10 mm and 100 mm when measuring normal transmittance value of filtered samples, and the path is 10 mm when measuring total transmittance value of non-filtered samples. The color differences obtained from the measurements are referred to as color intensity $(S)_{10}$, color intensity $(S)_{100}$ and color intensity (T) respectively. The reason why the light path is limited to 10 mm when measuring the total transmittance values is the technical difficulty to capture the scattered light when the light path is long.

Fig. 2 shows examples of results of measuring the relationship between color intensity and concentration for the Pt-Co color standards and the various dye solutions. The dyes in Fig. 3 and 4 are hydrophilic ones, and thus color intensity (T) and color intensity $(S)_{10}$ are the same. The relationship between dye concentration and color intensity assumes a form resembling the exponential function as expected by equation (3). However, colors of bright hues such as yellow and green, and colors of comparatively dark hues including purple, blue and red show a slight difference in trend. With colors of dark hues color intensity reaches maximum value at comparatively low concentration. When the concentrations are same, color intensity of dye solution with darker hue is larger than that with brighter hue. Although this seems to be reasonable, there still is some room for further studies on to what extent it coincides with the impression received by visual sense.

Fig.-2 Color Intensity(S) of Pt-Co Color Standards

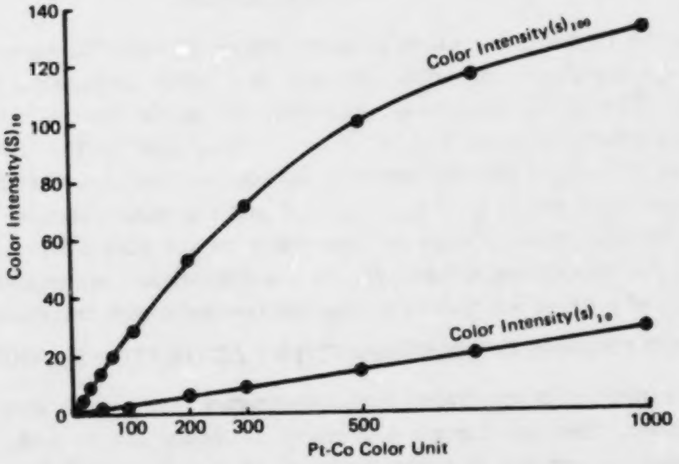


Fig.-3 Relationship between Dye Concentration (Yellowish Dye) and Color Intensity

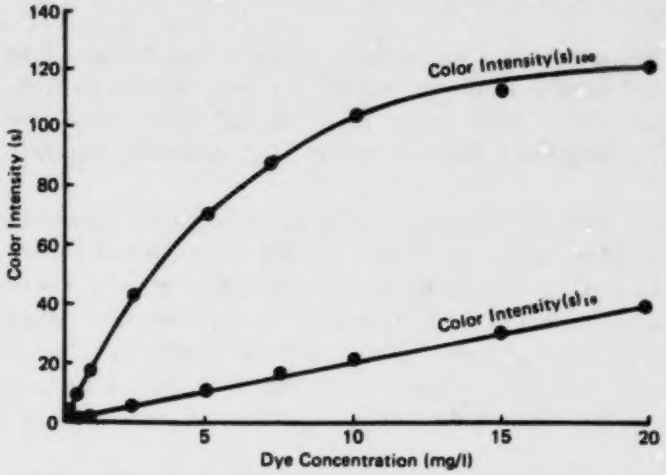
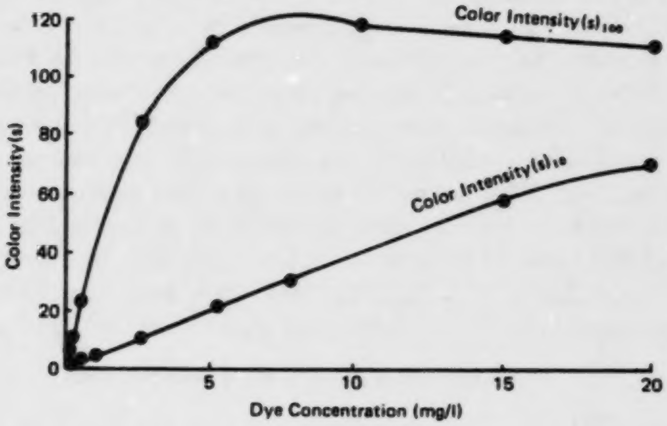


Fig.-4 Relationship between Dye Concentration (Violet Dye) and Color Intensity



At about 10 different stations in rivers flowing through the city of Hamamatsu, repeated measurements of color intensity and visual impression surveys were conducted. The visual impression tests were of simple nature, categorizing the impressions into three ranks, that is, 'clean', 'normal' and 'dirty'. The average color intensity of river water was calculated for the stations evaluated visually as 'normal'. The average color intensity $(S)_{100}$ was 2.5 and the color intensity (T) was 0.7. Although the impression of color of river water on site differs depending on water depth and the surrounding conditions, the abovementioned values give a rough idea on the limit of color of not making people feel uncomfortable and repugnance.

3. COLOR CHANGE OF WASTEWATER BY ACTIVATED SLUDGE PROCESS

With regards color removal of sewage containing textile wastewater by activated sludge process, there are reports confirming decolorization to some extent while others report no removal of color. Types of dyes and measurement methods are considered to have significant effects. Laboratory experiments were carried out in

Hamamatsu City to investigate color change of wastewater by activated sludge process using overall textile wastewater and process effluent containing specific type of dye.

Hamamatsu is famous for its production of textile goods with dye works of all types in the city. At the Chubu Sewage Treatment Plant that covers the major portion of the city, an average 109,400 m^3/day of sewage was handled in 1975. Nearly 20 percent or roughly 20,000 m^3/day (including textile process effluent of 14,720 m^3/day) of wastewater was from textile plants. There are 29 textile plants within its service area of which 28 are located in the drainage area of one of the pumping stations, the Minami Pumping Station. The Minami Pumping Station handles about 60,000 m^3/day of sewage of which nearly one-third are wastewater from textile plants. Excluding the non-colored wastewater generated at scouring and bleaching processes, colored wastewater accounts for roughly 24 percent at the Minami Plant. The quantity of dyes (including pigments) used by the previously mentioned 28 plants was about 27.3 tons per month, and as Table 1 indicates, pigments and reactive dye shared more than half of the total. When pigments are excluded, reactive dye accounted for over 50 percent of the total. Table 2 shows the percentage of dyes used in Japan in 1975. Dispersive dye is predominantly used as a whole.

Table-1 Percentage of Dyes Used in the Drainage Area of the Minami Pumping Station, Hamamatsu City

Sulphide	Leuco	Naphtolic	Dispersive	Reactive	Acid	Direct	Mordant	Basic	Pigment
6.7	9.0	4.0	0.4	35.1	2.8	2.9	0.8	0	38.3

Table-2 Percentage of Dyes Used in Japan

Sulphide	Leuco	Naphtolic	Dispersive	Reactive	Acid	Direct	Mordant	Basic	Pigment
6.2	7.1	4.3	39.6	9.1	6.6	5.5	2.6	3.1	15.9

Table-3 Color Intensity of Textile Wastewaters and Others

	Color Intensity$(S)_{100}$	Color Intensity$(S)_{10}$	Color Intensity(T)	Color Intensity$(S)_{10}$ Color Intensity(T)
Reactive Dye Waste	85.0 (35.9~110)	64.3 (4.3~89.1)	66.1 (4.8~90.8)	95.5% (89.6~98.2)
Sulphite Dye Waste	59.8 (9.7~78.9)	9.6 (0.8~14.8)	37.7 (27.3~53.5)	26.4% (29.~49.3)
Dispersive Dye Waste	73.3 (20.0~100)	16.1 (3.3~34.1)	38.5 (13.0~59.0)	39.4% (8.2~57.8)
Basic Dye Waste	122 (77.8~140)	65.3 (11.6~113)	75.4 (18.5~116)	82.5% (60.5~99.5)
Night Soil	102 (101~103)	64.6 (45.1~79.9)	86.6 (85.2~88.7)	73.2% (52.1~87.0)

The upper figures are the average values and the lower figures show the range.

3.1 EXPERIMENTAL METHOD

3.1.1 Samples Used in the Experiments

The activated sludge treatment experiments conducted here were to examine removal of color from municipal sewage containing comparatively small amount of textile wastewater.

Samples of domestic sewage for the experiment were taken at the Enshuhama Sewage Treatment Plant in Hamamatsu. This treatment plant is a small one, handling only domestic sewage with an average flow of 2,500 m^3/day. The population served was 10,600 as of the end of 1975. According to the result of water quality tests during a whole day, the color intensity and other water quality parameters such as BOD and SS have shown almost identical time variations by reaching the maximum at between 8 and 9 am. The range of color intensity $(S)_{100}$ variation during a day was from 5 to 25. As the samples collected at around 11 am were considered to show an average color intensity, the influent taken at 11 am was used as domestic sewage to be mixed with textile wastewater for the experiment.

As for the samples of textile wastewater, effluent containing various types of dyes altogether and that containing specific types of dyes were used. The former was collected at the aforementioned Minami Pumping Station while the latter was collected from actual dyeing plants.

The range of color intensity variation of sewage at the Minami Station during a daytime was 20 – 25, marking its peak at between 4 and 6 pm. Thus, sewage collected at 4.30 pm was used as the samples for the experiment.

Concerning the samples containing single type of dye in each sample, on the other hand, reactive, sulphide, dispersive and basic dyes were selected as types of dyes, and wastewaters from actual textile plants were collected as the samples except for basic dye. Since there are no plants in Hamamatsu using this type of dye, synthetic wastewater by dissolving basic dye was prepared. Table 3 shows the color intensity of each wastewater used in the experiment. In the Table, the upper figures show the average values while the lower figures indicate the range.

The ratio of color intensity $(S)_{10}$ to color intensity (T) serves as an index indicating the proportion of the color intensity caused by dissolved substances. It is quite natural that the reactive and basic dyes that are hydrophilic show larger values while hydrophobic sulphide and dispersive dyes show smaller values.

In some sewage treatment plants, night soil addition at the sewage treatment plant can be considered to cause coloring of treated water. Therefore, decoloring experiments were also conducted by using samples which were prepared by adding night soil to the domestic sewage. Color intensity of night soil is also shown in Table 3. It shows that night soil, too, has tremendously high color intensity.

3.1.2 Experimental Run, Experimental Apparatus and Experimental Conditions

Twelve experimental runs were carried out. In which, five runs are for textile wastewater containing various kinds of dyes altogether (hereinafter, these runs are called 'Mixed Dyes Waste'). For these runs, feed waters were prepared by adding the sewage taken at the Minami Pumping Station to domestic sewage so that ratios of the textile wastewater were 1, 2, 4, 8 and 16%, respectively. Four runs are for textile wastewaters, each containing reactive, sulphite, dispersive and basic dyes separately. In these runs, percentage of textile wastewater in the feed water was 10%. In the experimental run for night soil, one percent of night soil was added to the domestic sewage to prepare the feed water. And, as controls, domestic sewages collected at 9 am and 11 am were prepared for two different runs. Table 4 summarizes the water quality of each feed water influent for each experimental run. Due to technical difficulties in the measuring devices, some figures are missing in Color Intensity (T).

Table-4 Influent Water Quality of Each Experimental Run of Activated Sludge Treatment

Name of Run	Turbidity	SS (mg/)	COD (mg/)	Color Intensity(S)$_{100}$	Color Intensity(T)
Mixed Dyes Waste 1%	82 (50~175)	81.6 (57.0~108)	44.1 (31.7~53.5)	15.9 (13.4~20.6)	
Mixed Dyes Waste 2%	92 (50~250)	78.5 (48.8~146)	43.6 (30.4~62.1)	14.0 (9.9~19.5)	
Mixed Dyes Waste 4%	89 (33~190)	80.7 (45.0~190)	42.0 (27.1~52.5)	15.1 (10.6~23.6)	
Mixed Dyes Waste 8%	104 (53~220)	94.9 (60.8~218)	47.8 (37.6~70.6)	18.7 (13.2~37.1)	
Mixed Dyes Waste 16%	81 (22~160)	93.0 (39.1~214)	46.3 (35.3~62.7)	24.8 (12.9~42.3)	
Reactive Dye Waste 10%	114 (51~185)	89.1 (46.0~139)	50.8 (39.6~60.7)	74.1 (12.9~92.6)	15.4 (3.2~22.0)
Sulphite Dye Waste 10%	66 (37~168)	73.8 (52.7~108)	40.4 (32.4~47.9)	15.4 (10.2~22.2)	5.6 (3.6~7.9)
Dispersive Dye Waste 10%	162 (47~368)	173 (73.0~269)	82.0 (38.7~117)	23.5 (15.5~54.7)	8.9 (4.0~27.5)
Basic Dye Waste 10%	108 (45~178)	106 (69.0~150)	48.1 (36.4~56.2)	24.5 (9.3~102)	13.3 (5.3~48.3)
Night Soil 1%	156 (60~382)	124 (67.0~335)	64.3 (47.5~113)	25.1 (20.7~32.6)	5.4 (3.1~11.5)
Domestic Sewage Taken at 9 AM	99 (70~230)	122 (7.0~168)	52.1 (41.8~61.2)	17.3 (14.1~24.2)	
Domestic Sewage Taken at 11 AM	102 (42~245)	94.7 (25.0~171)	39.7 (19.4~65.1)	13.9 (11.1~18.8)	

The upper figures are the average values and the lower figures show the range.

Table 3 and Table 4 show some interesting points when compared. In the case of basic dye, after being mixed with domestic sewage, coloring caused by dissolved substances dropped markedly. Comparing with the case of reactive dye which is also hydrophilic, color intensity (S) of basic dye wastewater before mixing with domestic sewage is a little larger than that of reactive dye wastewater. But after the wastewaters are mixed with domestic sewage, the color intensity (S) of basic dyes become much smaller. This is reasoned to the fact that as the basic dye wastewater is not a real wastewater from plants and a reasonable amount of basic dyes are absorbed by the SS in the domestic sewage. Therefore, in such decoloring treatment experiments, there is a danger of acquring misleading results when not employing actual wastewater from textile plants. The apparatus used for the experiment was a laboratory scale one with primary sedimentation tank of 4 liters, aeration tank of 18 liters, and final settling tank of 7.5 liters. Experimental conditions are shown in Table 5. The feed water was prepared every day from the samples taken on the same day or one day before. The feed water was mixed slowly and continuously in the storage tank to prevent sedimentation of suspended matters in the raw water.

Table-5 Experimental Conditions

Flow Rate	55 ml/min.
Settling Time of Primary Settler	1.2 hrs.
Aeration Time	5.5 hrs.
Settling Time of Secondary Settler	2.3 hrs.
Water Temperature in Aeration Tank	20°C
Experimental Period in Each Run	2 weeks

As for seed sludge of activated sludge, the activated sludge at the Chubu Sewage Treatment Plant was used for the experimental runs with textile wastewater. With regards runs with night soil and domestic sewage, the activated sludge of the Yayoi Sewage Treatment Plant in Hamamatsu that exclusively handles domestic sewage was used. The experimental period for each run was 2 weeks. With regards experimental runs with specific dyes, two weeks of acclimation period were given for each run. No acclimation period was given for other runs since water quality of feed water was similar to that of the treatment plant where seed sludge was collected. The raw sludge at the preliminary settling tank was withdrawn arbitrarily depending upon the generation of sludge. The adjustment of return sludge and the withdrawal of the excess sludge were conducted at the secondary settling tank by determing the amount based on the results of measuring the SV, MLSS, etc. of the aeration tank.

The water temperature in the aeration tank was maintained at 20°C throughout the experimental period.

The water quality of feed water and treated water was measured daily. pH, SV, and DO in the aeration tank was also measured daily. MLSS was measured 2 to 4 times during the experimental period.

3.2 EXPERIMENTAL RESULTS

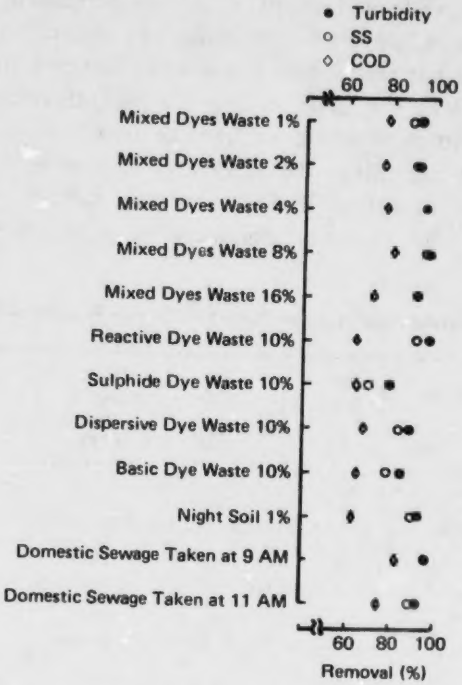
Table 6 shows DO, MLSS, SVI and BOD loading in the aeration tank for each experimental run. The BOD loading indicated in the table is a rough estimate which is calculated from COD of feed water since measurement of BOD in feed water was not performed. Fig. 5 shows the removals of COD, SS and turbidity. According to Fig. 5, the removals of SS with regards sulphide and basic dyes were slightly bad. However, SVI in these runs was smaller when compared with runs of the mixed dyes waste and bulking was not observed. Rather, in runs of mixed dyes waste there was a tendency of causing bulking. The COD removal was about 65 percent in runs of specific dye waste and about 75 to 80 percent in runs of domestic sewage and mixed dyes waste with low ratios. In general, the COD removal because low when the textile wastewater was mixed. However, in the experimental runs with low removal rate, the BOD loading was high, therefore it was not clear whether the difference in treatment conditions or textile wastewater had caused the drop in the removal rate. Since the aim of the experiment was to investigate the decolorization effect of activated sludge process, further examinations were not conducted because the treatment condition of each experimental run was judged fairly satisfactory.

Table-6 Operating Parameters of Activated Sludge Process of Each Run

Name of Run	pH	DO (mg/)	SV (%)	MLSS (mg/)	SVI	BOD Loading (kg/SS·kg/d)
Mixed Dyes Waste 1%	7.0 (6.7~7.3)	0.9 (0.3~1.9)	19.3 (9.0~31.5)	1330 (1230~1420)	145	0.19
Mixed Dyes Waste 2%	7.2 (7.0~7.4)	0.9 (0.3~4.5)	20.6 (4.5~31.0)	1200 (1070~1330)	170	0.21
Mixed Dyes Waste 4%	7.2 (7.1~7.4)	0.4 (0.3~0.6)	27.6 (14.5~47.0)	1220 (850~1860)	225	0.20
Mixed Dyes Waste 8%	7.1 (6.2~7.3)	0.8 (0.5~1.6)	24.2 (10.5~48.0)	1490 (1170~2300)	160	0.19
Mixed Dyes Waste 16%	7.2 (6.8~7.6)	1.1 (0.3~4.1)	25.2 (3.0~73.5)	1740 (1400~2400)	145	0.15
Reactive Dye Waste 10%	7.4 (7.1~7.6)	1.2 (0.3~3.1)	13.0 (5.0~24.0)	560 (380~780)	174 (119~212)	0.52
Sulphide Dye Waste 10%	6.6 (5.7~7.2)	1.5 (0.4~2.9)	8.0 (5.0~14.0)	1320 (880~1790)	64 (48~78)	0.18
Dispersive Dye Waste 10%	6.8 (6.2~7.2)	1.8 (0.8~5.0)	12.0 (7.0~15.5)	1110 (980~1270)	99 (78~134)	0.42
Basic Dye Waste 10%	7.3 (6.9~7.8)	3.2 (0.9~6.9)	13.0 (7.0~24.0)	1060 (600~1590)	117 (98~145)	0.26
Night Soil 1%	7.6 (7.4~7.9)	1.5 (0.2~3.1)	11.5 (2.0~28.0)	1030 (970~1150)	141 (82~183)	0.36
Domestic Sewage Taken at 9 AM	7.0 (65.~7.0)	0.7 (0.3~1.6)	18.2 (9.0~30.5)	1540 (1430~1660)	120	0.20
Domestic Sewage Taken at 11 AM	7.4 (6.1~7.0)	1.5 (0.4~4.7)	12.6 (0~24.0)	1560 (1380~1740)	80	0.15

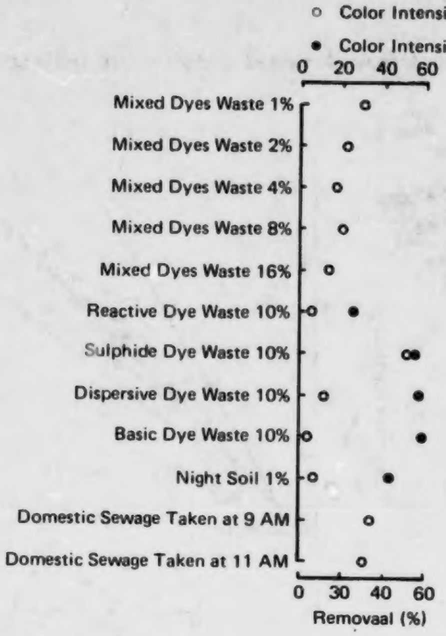
The upper figures are the average values and the lower figures show the range.

Fig.-5 Removal of Turbidity, SS and COD in Each Experimental Run of Activated Sludge Treatment



The removal of color intensity in each experimental run is summarized in Fig. 6. Since color intensity is not a linear index, the concept of removal may not be appropriate. But for convenience sake the removal was defined as the ratio of the difference of color intensities of the feed water and the treated water to the color intensity of the feed water. The removal of color intensity (T) of runs of mixed dyes waste and domestic sewage are not listed owing to malfunctioning of measuring devices.

Fig.-6. Removal of Color in Each Experimental Run of Activated Sludge Treatment



The removal of color intensity $(S)_{100}$ was about 30 to 40% for runs of domestic sewage. But when textile wastewater was mixed, the removal was about 20% with regards runs of mixed dyes waste and below 10% concerning runs of specific dye waste excluding that of sulphide dye waste. With regards sulphide dye waste, substances classified as dissoluble were predominantly in the form of colloid particles and were absorbed by the activated sludge, thus giving higher removal. Furthermore, it is noteworthy to mention that sewage added with night soil was very low in removal compared to sewage containing only domestic sewage.

Also, concerning removal of color intensity (T), it was about 60% for the hydrophobic dispersive and sulphide dyes with the apparent color by suspended matters considered totally removed. Naturally, the removal of color intensity (T) for the hydrophilic reactive dye waste water was as low as 25%. With regards basic dye waste water which is also hydrophilic, the removal of color intensity (T) was extremely high owing to the reasons mentioned before that actual waste water from textile plant was not used. In short, as it was a synthetic wastewater by artifically dissolving dyes in water, the dyestuffs were adsorved by the SS or activated sludge in the water and thus the apparent removal rate was considered to be high. When actual wastewater from textile plant is used, it is assumed that the removal of color intensity (T) is extremely small.

To further study on the removal of color caused by dissolved substances, chromaticity diagrams were made for samples of filtered feed water and treated water (0.45 μ membrane filteration). Figures 7 to 10 show the chromaticity diagrams for each experimental runs of sewage containing only domestic sewage, sewage with 16% mixed dyes waste, sewage containing reactive dye and sewage added with dispersive dye.

Fig.-7. Change in Color through Activated Sludge Process: Influent is the domestic Sewage

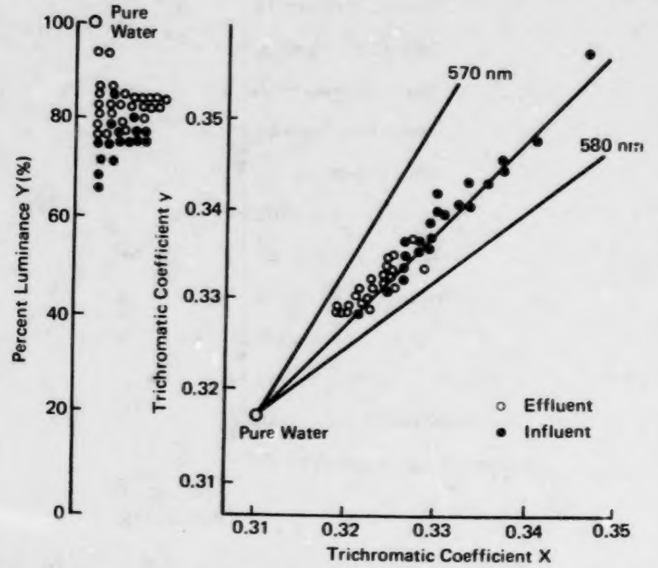


Fig.-8 Change in Color through Activated Sludge Process: Influent is the Sewage containing 16% of textile waste

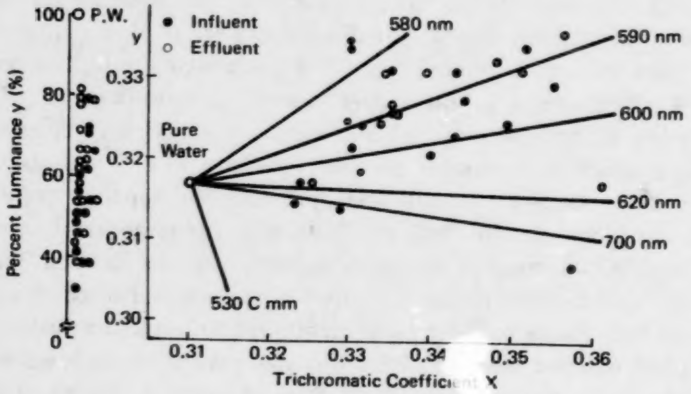


Fig.-9 **Change in Color through Activated Sludge Process: Influent is the Sewage containing 10% of reactive dye waste**

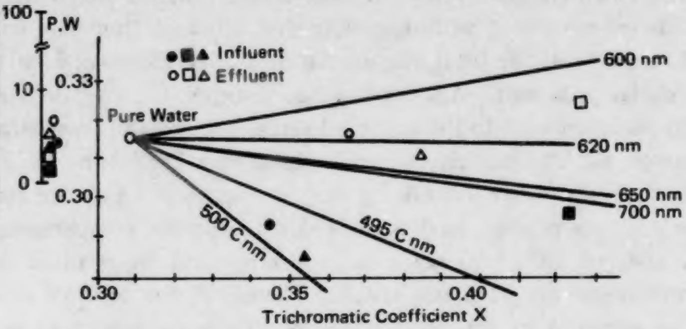


Fig.-10 **Change in Color through Activated Sludge Process: Influent is the Sewage Containing 10% of dispersive dye waste**

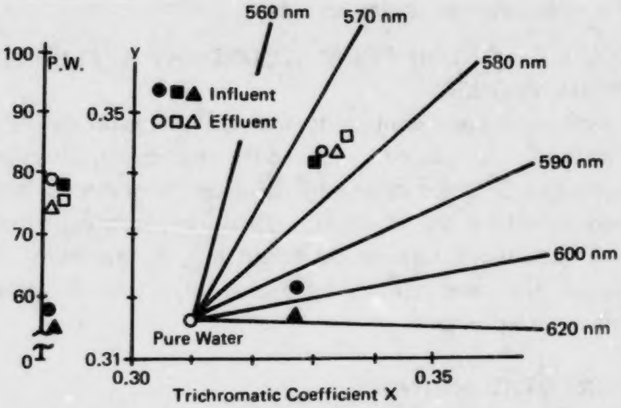


Figure 7 shows that the dominant wave length for both feed water and treated water containing only domestic sewage was 57t nm (yellow). And by applying the activated sludge process, the excitation purity has decreased while the luminance has increased, testifying to the fact that decoloring was evident.

On the other hand, Figure 8 shows that the luminance followed a trend of increase with regards sewage containing 16% of mixed dyes. And also the excitation purity followed a similar trend. In this experimental run, considering the fact that 80% of the sewage was domestic sewage, color by dissolved substances of the dyestuffs is considered to be strengthened by the activated sludge process. With regards this point, Figures 9 and 10 on reactive and dispersive dyes make the matters clear. Concerning the run of reactive dye waste, the hues of red and purple are treated and converted to red color in the neighborhood of 620 nm. The excitation purity, too, increased slightly. Furthermore, in case of the run of dispersive dye waste, even when the hue of feed water is either red or yellow, the hue of treated water changes to yellow (about 575 nm) with the excitation purity increasing greatly.

When processing sewage containing only domestic sewage, the excitation purity drops when treated by the activated sludge process. But as mentioned above, with regards sewage mixed with dispersive dye, although domestic sewage occupies about 80 to 90 % of the total, the excitation purity increases when treated by the activated sludge treatment. Also as the hue changes, the dyestuffs are considered to undergo changes owing to the activated sludge process and show stronger color.

Although the chromaticity diagram for sewage containing 1% of night soil is similar to that of sewage containing only domestic sewage, the rate of drop in excitation purity is smaller. And as understood from the comparisons made in Fig. 6, by the addition of 1% of night soil color removal drops to about 1/4 of the sewage containing only domestic sewage. Therefore, the color of domestic sewage that can be removed by activated sludge process is considered to be only that of miscellaneous drainage and not night soil.

Although not illustrated, the chromaticity diagram of the run of sulphide dye waste is nearly identical to that of domestic sewage, proving that sulphide dye can be removed by the activated sludge process.

4. REMOVAL OF COLOR FROM SECONDARY EFFLUENT BY PHYSICAL CHEMICAL PROCESS

When treating sewage containing textile wastewater by the activated sludge process, removal of color caused by suspended matters is quite effective. But, on the other hand, removal of color caused by dissolved substances is poor and sometimes color is strengthened by the treatment. Therefore, in order to effectively remove color by such substances, tertiary treatment may be necessary. Thus studies were made to examine the color removal by ozonation, activated carbon adsorption and chemical coagulation processes.

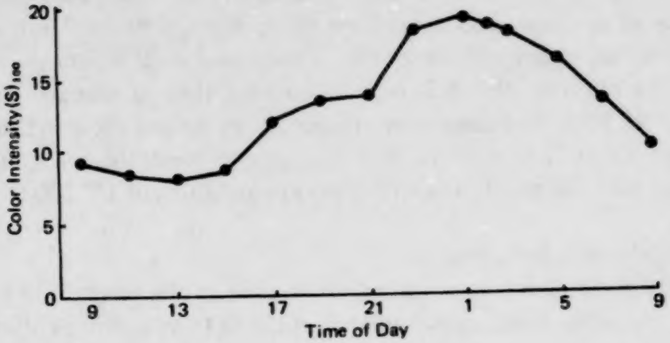
4.1 EXPERIMENTAL METHOD

4.1.1 Samples Used for the Experiments

Similar to the activated sludge process experiment, the experiments were conducted by preparing samples containing various types of dyes altogether and containing only specific dyes. As the former samples the secondary effluent (before chlorination) at the Chubu Sewage Treatment Plant in Hamamatsu city were used, and treated water from the activated sludge experiments was used for the latter samples.

As mentioned previously, about 20% of the influent at the Chubu Plant is wastewater from textile plants (about 13% when only colored wastewater), thus the effluent is considerably colored. Fig. 11 shows an example of the daily variation of color intensity $(S)_{100}$ of the secondary effluent. Since visual color evaluation of river water mentioned before showed that a rough boundary between 'dirty' and 'clean' is about 2.5 in color intensity $(S)_{100}$, the secondary effluent of the plant is colored to a good extent. The dominant wave length of light absorption was between 570 and 600 nm with the hue being yellowish orange.

Fig.-11 **Diurnal Variation of Color Intensity$(S)_{100}$ of the Secondary Effluent of the Chubu Sewage Treatment Plant**



4.1.2 Experimental Method

(1) Ozonation process

The experiment was based on the batch test with an ozone generator of maximum capacity of 0.9 g/hour. One-liter of sample was used for each experiment, and the reacted ozone was calculated from the following equation:

Reacted ozone (mg/1) = [Ozonation per unit time (mg/min) x Elapsed time (min) – Excess ozone (mg)]

The measurement of ozone contained in the water was conducted by titration of sodium thiosulfate.

(2) Activated carbon adosrption

The decolorization experiment by the activated carbon adsorption process was conducted by both the adsorption isotherm test and the continuous column test.

The experimental process of the isotherm test was as follows:

The activated carbon was crushed in the mortar, and only those passing through the 100 mesh seive were taken and dried for 2 hours in the drying oven adjusted at 105 – 110°C. 0.01, 0.025, 0.075 and 0.15 grams of the activated carbon were placed in different 500 ml Erlenmeyer flasks with stopper, respectively. Then the flasks were added with the water sample which had been filtered through 1 μ membrane filter, and then thoroughly shaked to be placed in a 20 C constant temperature room overnight. The color intensity $(S)_{100}$ of the water sample was measured prior to and after the adsorption test, and the difference was regarded as the color intensity adsorbed by the carbon. The adsorption test was repeated four or five times using the same water sample, and the arithmetical mean was obtained.

The activated carbons used in the adsorption isotherm test with the secondary effluent of the Chubu Plant were 12 types of 7 manufacturers. Concerning other secondary effluents, three types of activated carbons were used. The selection of three types of carbons were based on the test results with the Chubu plant effluent, and one type of carbon was chosen from each category evaluated as 'good', 'normal' and 'poor'. The 12 types of activated carbon were all of coal type of which four were spherical carbons and the rest granular carbons.

The activated carbon column used in the continuous test was made of acrylic resin with an inner diameter of 34 mm and length 2 m. The operation of the carbon column was of a pressurized down-flow type. The continuous test was held only on the filtered secondary effluent of the Chubu treatment plant.

First, to examine the influence of contact time, a single type of activated carbon was selected. And tests were conducted by setting the condition at LV 100 (m/day) and SV at 2, 4, 6, 8, 10, and 12 (1/hour). Next, the color removal by activated carbon was compared of the 12 types at conditions of LV 100 (m/day) and SV 4 (1/hour).

(3) Chemical coagulation process

The following chemical coagulants were used in the tests: Four types of metal coagulants including alum, polialuminum chloride (PAC), ferric chloride and lime. Twelve types of polymers including eight cationic and four anionic coagulants.

All tests were conducted by jar tests, using coagulants independently and in combination. The mixing were conducted in three different speeds as follow: rapid mix (150 rpm), medium mix (80 rpm) and flocculation (40 rpm). When single coagulant was used, the mixing time was 1.5 minutes for rapid mix, 0.5 minutes for medium mix and 3 minutes for flocculation. And when the coagulants were combined, the time was 3, 1 and 6 minutes respectively.

4.2 EXPERIMENTAL RESULTS AND DISCUSSION

4.2.1 Ozonation

Ozone's ability to remove color caused by organic substances is well known. In this experiment, it was also confirmed that the color removal by ozone of secondary effluent colored by textile wastewater produced extremely good results. Fig. 12 shows the relationship between the amount of reacted ozone and the color intensity $(S)_{100}$ removal. Here too, the removal is defined as the ratio of the difference in color intensity of the feed water and the treated water to the color intensity of the feed water for convenience sake.

Fig.-12 Removal of Color by Ozonation: Samples are filtered secondary effluent of the Chubu Sewage Treatment Plant

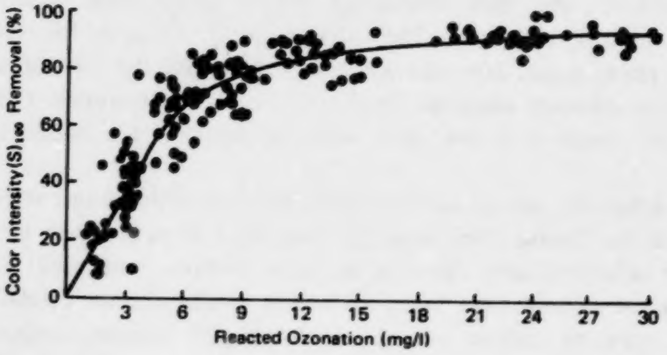


Fig. 13 similarly shows the change in color by ozonation on the chromaticity diagram. It shows that with the increase in reacted ozone, the luminance increases and the excitation purity decreases proving that decolorization is progressing. Also, the dominant wave length changes from 588 nm to 577 nm, and the orange hue of the feed water changes to a yellowish hue.

Fig.-13 Change in Color by Ozonation; The sample is filtered secondary effluent of the Chubu Sewage Treatment Plant. Figures in the parentheses show reacted ozone in mg/

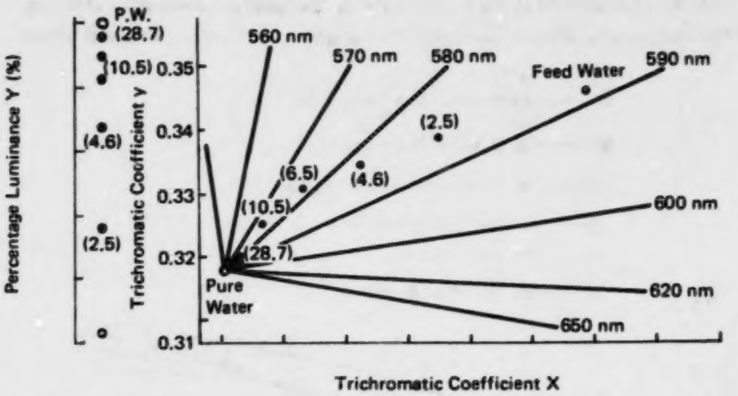
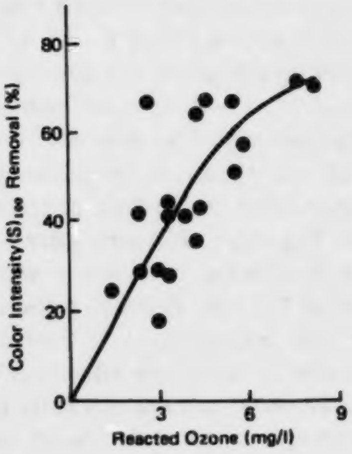


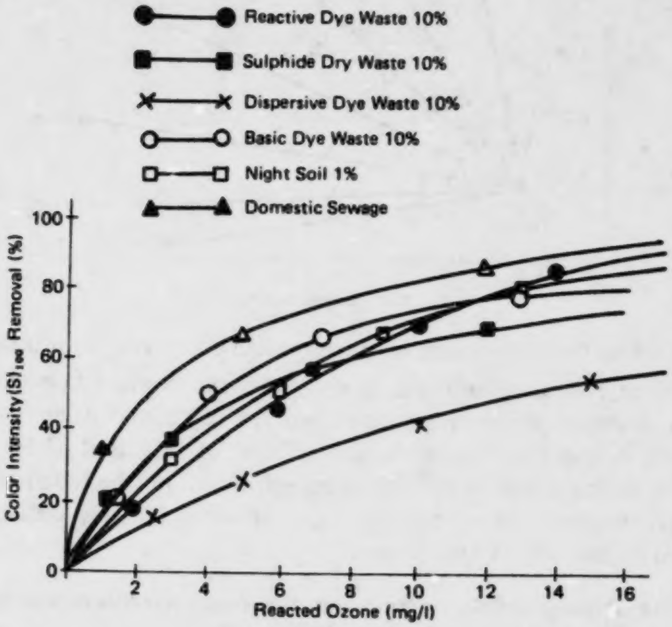
Fig. 14 shows the relationship between reacted ozone and color intensity (S)100 removal with regards non-filtered secondary effluent of the Chubu Sewage Treatment Plnat. It shows similar results as shown in Fig. 12, and when ozone of low concentration is dosed to the secondary effluent of this level of SS (roughly 20 mg/l), ozone consumption or its decoloring effect will not be much affected. Also concerning removal of color intensity (T), similar results were obtained although the deviation of data were a little larger.

Fig.-14 Removal of Color by Ozonation: Samples are non-filtered secondary effluent of the Chubu Sewage Treatment Plant



As to the results of the runs of specific dyes, the relation between reacted ozone and removal of color intensity $(S)_{100}$ was as shown in Fig. 15. In Fig. 15, results of runs of 1 percent night soil addition and domestic sewage alone are also shown for comparison. The figure indicates that except for the extremely poor decoloring effect of dispersive dye, there were no significant differences observed of other dyes. However, they seem to be slightly inferior when compared with secondary effluent of domestic sewage.

Fig.-15 Removal of Dissolved Color by Ozonation: Samples are secondary effluents obtained by treatment of sewage containing specific kinds of dye and others



Although not illustrated, relationship between color intensity (T) removal and reacted ozone was found to be similar to that shown in Fig. 5. In this case, however, not only dispersive dye but also sulphide dye had a poor color removal. Perhaps, this is reasoned to the fact that as both dyes are hydrophobic, oxidation by ozone is not effective in removing color caused by suspended matters.

From Figures 12, 14 and 15, it can be commonly said that decolorization progresses roughly in proportion to the amount of reacted ozone until the removal reaches 60 or 70 percent. The color remaining after ozonation, as an example of which is shown in Fig. 13, is yellowish in most case with the dominant wave length being in the neighborhood of 575 nm. It is well known that yellowish color often remains when wastes of high concentration is treated by ozone. Therefore, the residual color after ozonation of secondary effluent might be caused by oxidation products of dissolved dye. However, since similar color remains in the ozone treated effluent when domestic sewage, or domestic sewage and night soil were handled it was considered that remaining color was caused mainly by night soil.

4.2.2 Activated Carbon Adsorption

The adsorption at equilibrium state of dissolved organic substances by activated carbon may be expressed by the following Freundich Equation:

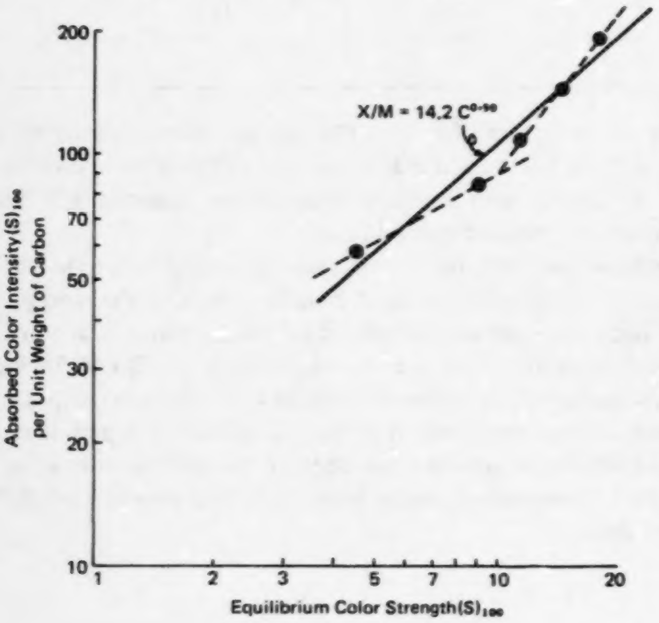
$$X/M = KC$$

Here, X is the amount of adsorbed substances. M is the amount of activated carbon. C is the equilibrium concentration after adsorption. And K and n are constants.

When applying adsorption of color in the equation, concentration of the color causing substances or a parameter which has a linear relationship with the substance should be used. But since there is no proper means of measuring the concentration of color causing substances, the color intensity $(S)_{100}$ was incorporated to obtain approximate values.

Adsorption isotherm tests were carried out using the secondary effluent of the Chubu sewage treatment plant and 12 types of activated carbons, A to L. In some cases, straight lines were obtained. In most cases, however, knicks appeared and two straight lines gave closer approximation than single line. An example is shown in Fig. 16. As mentioned before, this is caused by the fact that the adsorption expressed by color intensity is not in proportion to the concentration of the color causing substances and also the substances different in adsorption characteristics were present.

Fig.-16 Adsorption Isotherm, Carbon A



However, to make rough comparisons, it was judged that single line was possible for approximation, and, thus, K and l/n values of the equation were calculated from the data obtained by the experiments using the secondary effluent of the Chubu Plant. The result is shown in Table 7. It indicates that the adsorption ability differs greatly depending on the types of carbons. Activated carbons H, E, J, I and G have shown high decoloring efficiency while carbons L, K and B were poor.

Table-7 Constants in Fraundlich Equation: Adsorption of Color from Secondary Effluent containing Mixed Dyes

Name of Carbon	K	l/n
A	14.2	0.90
B	6.8	1.20
C	12.2	0.75
D	12.0	0.80
E	62.0	0.45
F	4.4	1.10
G	45.0	0.60
H	100	0.40
I	46.0	0.55
J	64.0	0.50
K	11.0	0.90
L	2.9	1.20

Based on the assumption that the average color intensity $(S)_{100}$ of the feed water is 22 from Fig. 11 and that 80 percent of this is to be removed, the necessary quantities of carbon with the largest adsorption capacity and with the smallest adsorption capacity differ about 10 times.

Adsorption isotherm tests with secondary effluent of the sewage containing 10 percent of textile wastewater of specific dyes and the sewage containing one percent of night soil were also conducted by using carbons E, H and L. K and l/n values obtained from the tests are shown in Table 8. The difference in activated carbons had similar effect to those obtained from the tests using secondary effluent of the Chubu treatment plant. However, it should be noted that the adsorption characteristics differred greatly dependent on the difference in dyes. Reactive dyes were adsorbed comparatively easily while basic dyes proved most difficult to adsorb of all tested dyes.

Table-8 Constants in Freundlich Equation: Adsorption of Color from Secondary Effluent containing Specific Types of Dye and Others

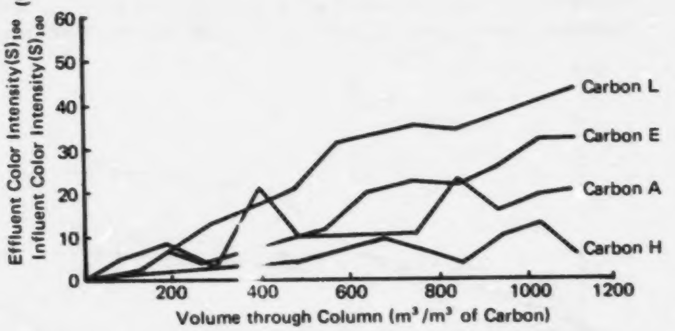
Name of Carbon / Name of Dry	K			l/n		
	E	H	L	E	H	L
Reactive	123	177	0.45	0.21	0.18	1.51
Sulphide	22.2	38.0	0.25	0.89	0.70	0.72
Disperse	9.7	26.6	0.14	1.19	0.87	2.48
Basic	26.0	22.2	0.83	1.28	1.48	1.61
Night Soil	52.0	62.0	8.0	0.42	0.48	0.78

The effect of contact time was investigated by continuous column test using the carbon A. The filtered secondary effluent of the Chubu Plant was used as the feed water. As a result, as far as color intensity removals at the same feed water volume applied to carbon volume was concerned, the removal rate was obviously large when SV = 2 (1/hour, hereinafter the same). However, within the SV range of between 4 and 12, there was no significant difference.

Furthermore, to confirm the results obtained from the adsorption isotherm tests, continuous tests were conducted on all activated carbon employed in the experiment by using the filtered secondary effluent of the Chubu Plant as feed water.

In this instance, the LV was set at 100 (m/day) and SV at 4 (1/hour). The results of the slected activated carbons are shown in Fig. 17.

Fig.-17 Comparison of Adsorptive Capacities of Test Carbons for Color Strength$(S)_{100}$ Influent is the secondary effluent of the Chubu Sewage Treatment Plant



From the adsorption isotherm test, activated carbon E was superior in adsorption characteristic to carbon A. However, data shown in Fig. 17 are not so. There were this kind of minor differences in the results of both tests, but as a whole, continuous column tests gave the similar results to those obtained from the adsorption isotherm tests.

The difference in adsorption capacity owing to the difference in activated carbons was compared with the characteristics of the carbons themselves. Table 9 shows the characteristics of activated carbons given by each manufacturer. The blank space in the Table is due to lack of information. Although the values listed in the Table are not so precise, the activated carbons having greater transitional pores can be judged to have higher adsorption capacity. Therefore, dyestuff substances that have undergone biological treatment are likely to be adsorbed in the transitional pores.

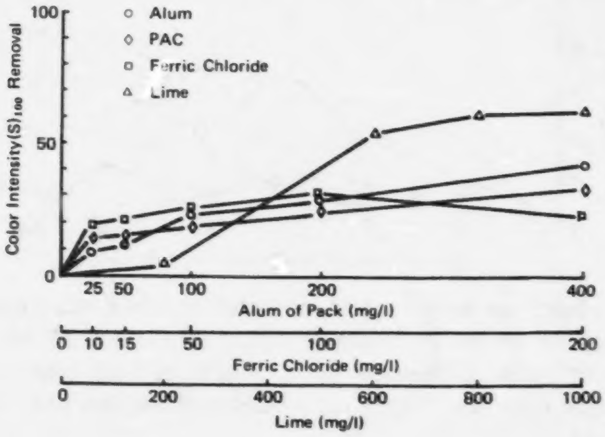
Table-9 Properties of Test Carbons

Test Carbon		A	B	C	D	E	F	G	H	I	J	L	K
Apparent Density	g/cc	0.48	0.44	0.45 -0.5	0.45 -0.5	0.4 -0.45	0.43	0.44	0.41	0.40	0.4 -0.5	0.49 -0.51	0.44 -0.51
Hardness	%			>98	>98	>90	97	90	91	99	>93	>99	88 -92
Ash	%	<8.0	<8.5				3.1	6.4	7.2		<10	15 -16	3 -5
Iodine Adsorption Cap.	mg/g	>900	>1000	900 -1150	900 -1150	>350	1080	924	785	1014	>1000	990	1000 -1100
Methylene Blue Ads. Cap	ml/g											160	200 -220
Benzene Ads. Cap	%						36.8	28.4	20.3	41.2	>28		
Specific Surface Area	m^2/g	>950	1000 -1100	900 -1200	900 -1200	900 1200	11001	1070	750		>1000	900	
Approximate Pore Size Distribution	20 Å cc/g	0.12										0.15	0.20
	$20\text{-}10^3$ Å cc/g	0.22					0.24	0.85	0.92	0.92		0.32	0.27
	$10^3\text{-}10^4$ Å cc/g						0.48	0.43	0.48	0.30		0.42	0.11

4.2.3 Chemical Coagulation Process

Various chemical coagulants were used in the jar tests of the secondary effluent of the Chubu Plant toinvestigate on the decoloring efficiency. In general, only unsatisfactory results were obtained. As an example, the test results employing metal coagulants are shown in Fig. 18. The most effective was lime. However, in order to conduct effective removal, extremely high dose was needed proving impractical to serve only for color removal.

Fig.-18 Relationship between Coagulant Dose and Color Strength$(S)_{100}$ Removal: Samples are the secondary effluent of the Chubu Sewage Treatment Plant



With regards polymer, anionic coagulants were virtually ineffective. However, cationic coagulants, when added with over 10 mg/liter, more than 50% of color caused by dissolved substances was removed, but such a high dose was beyond practical use. Poor flocc settlability is another defect.

Any combinations of metal salts and polymers did not give satisfactory result too.

5. Summary and Conclusion

(1) As a result of examining the existing measurement methods of color, particularly color due to industrial wastewater, it is considered most appropriate to express color by color difference with pure water. This can be done by measuring the normal transmittance value of the filtered sample with regards color by dissolved substances, and by measuring the total transmittance value with regards suspended matters. The former is referred to as color intensity (S) and the latter color intensity (T).

Color intensity of river water and visual impression obtained by looking at the river were compared. Although the figures are rough, the limits that did not make people feel uncomfortable or repugnance was 2.5 in color intensity $(S)_{100}$ and 0.7 in color intensity (T), respectively.

(2) Experiments on activated sludge treatment of sewage containing about 10 percent of textile wastewater were carried out. The removal of color caused by suspended matters was quite effective, but removal of color caused by dissolved substances was generally poor, and color was sometimes strengthened.

Color caused by dissolved substances in domestic sewage was removed 30 – 40 percent by activated sludge process. However, 1 percent of night soil addition to the domestic sewage, decrease color removal rate considerably.

(3) Experiments on ozonation of secondary effluent was conducted to remove residual color due to textile wastewater. Color caused by dissolved substances was effectively removed by ozonation. About 60 and 70 percent removal was obtained at a dose of less than 10 mg/l. Removal of color was roughly proportional to dose up to this level. However, when it exceeded this level, the removal of color per unit of reacted ozone decreased considerably.

Color caused by suspended matters was not effectively removed by ozonization. Rather, the suspended matters in secondary effluent was irrelevant to ozone consumption.

(4) Similarly, experiments on activated carbon adsorption of color remaining in secondary effluent were conducted.

Adsorption by activated carbon depended greatly on the types of activated carbons and the types of dyes present in water. It was found that activated carbons with greater transitional pores were particulary effective to remove residual color. Within the range of the dyes experimented, color by reactive dye was comparatively well removed by activated carbon.

(5) Chemical coagulation was considered to be inadequate process when the purpose is only to remove residual color in the secondary effluent.

CHAPTER 8
DEVELOPMENT AND EVALUATION OF AUTOMATIC WATER QUALITY MONITORING EQUIPMENTS

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CHAPTER 8

DEVELOPMENT AND EVALUATION OF AUTOMATIC WATER QUALITY MONITORING EQUIPMENTS

I.CURRENT STATE OF THE DEVELOPMENT OF AUTOMATIC WATER QUALITY MONITORS FOR HAZARDOUS SUBSTANCES IN WASTEWATER

1. INTRODUCTION

When a public sewage system receives industrial wastewater, provisions should be taken to prevent possible adverse effects on facilities or treatment function. It should not cause any difficulty in meeting the effluent standards applied to the treatment plant, nor create particular difficulty in treatment and disposal of sewage sludge. For these purposes, ordinances concerning pretreatment standards are provided to regulate the quality of wastewater being discharged into a sewerage system. Especially with regards to hazardous substances, pretreatment standards can be, and in most cases are, as stringent as the effluent standards applied to industrial wastewater discharged directly into public bodies of water.

To effectuate such ordinances and regulations, it is important not only to thoroughly guide and supervise the pretreatment facilities of the factories but also to establish a system to monitor water quality of industrial wastewater being discharged, which the municipalities across the nation are presently eneavoring. However, by a manual monitoring system, there are limits in the frequency of city officials inspecting all of the industrial plants, and there are danger of overlooking the discharge of harmful wastewater either by accident or on purpose. On the other hand, the current status of automatic monitors is that there are only few automatic monitors put to practical use to check wastewater being discharged. And even those put to practice are complicated in structure and require considerable manpower for operation and maintenance. Therefore, installation of these equipments, if they are installed, are limited to the places such as sewage treatment plants and pumping stations. However, automatic monitors that are most desirable to be developed are those highly reliable and free from maintenance, for example, similar to flight recorders mounted on aircraft and those which can readily be installed in sewers or outlets of industrial wastewater into sewers. For the past few years, the Ministry of Construction have entrusted the Association of Electrical Engineering to conduct researches to boost development of such automatic water quality monitors for hazardous substances.

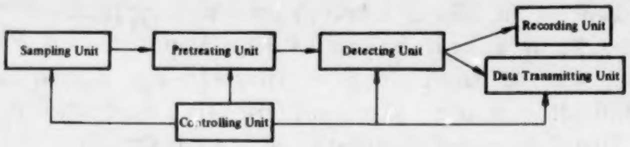
This report summarizes on the present situation of the development of automatic water quality monitors for hazardous substances and related problems.

2. PROBLEMS RELATED TO THE DEVELOPMENT OF AUTOMATIC MONITORS

2.1 OUTLINE

An automatic water quality monitoring device can be divided into several units as shown in Fig. 1. The structure and system of each unit differ depending on the characteristics of water handled and substances to be monitored, thus the problems involved are varied. Even when water is comparatively clean and the constituents are relatively simple, automatic measurement of hazardous substances is not easy. Since sewage contains wide variety of substances, most of which interfere the measurement, and have the characteristics to easily develope biological slime, automatic monitoring of harmful substances in sewage is quite difficult.

Fig.-1 Schematic Configulation of Automatic Water Quality Monitor



Major problems involved are as follows:

a) Suspended solids

The sampling pumps and valves in the piping are easily subjected to clogging in continuous operation as wastewater contains a good amount of suspended solids and floating substances. Normally, the substances to be measured in wastewater like heavy metals are contained in suspended solids. However, the method to detect and analyze substances is generally only possible when the substances are in dissolved form. Therefore, a process to dissolve substances contained in suspended solids becomes necessary. Even when this dissolving process is unnecessary, there are many instances requiring pulverization of suspended solids to obtain stable data.

b) Interferences

As substances found in wastewater are diversified and varied, when analyzing hazardous substances such as heavy metals, it is necessary to either eliminate interferences or mask them. Accordingly, the automatic monitoring devices must be equipped with a mechanism to eliminate the interferences. These mechanisms are hard to automate, and even when automation is possible the mechanical structures will be extremely complicated.

c) Biological slime

Wastewater usually contains a large amount of organics making it ideal for microorganisms to grow. Therefore, slime easily develops in the inner walls of the equipment coming in contact with the sample. Slime not only causes clogging of piping but also obstructs measurement by developing at the surface of the detector. Also, substances contained in water are often adsorbed on the slime to change quality of water, or slimes are often detached from the wall to interfere measurements.

2.2 SAMPLING UNIT

The engineering methods and equipments are available to take samples of wastewater directly from a sewer either in large rate continuous flow system or batch system. However, concerning sampling of extremely small continuous flow, the available methods and equipments are not considered satisfactory, since suction inlet and piping are liable to clog easily. Submersible pumps (with cutter) and air-lifts are employed for sampling continuous large rate discharge while vacuum suction sampling or air lifts are used for batch sampling. Generally, strainers of suction inlets are not useful and at times are harmful by accelerating clogging.

In general, there are two types of automatic monitors, that is, continuous type and batch type. In case of the continuous type, sampling of large rate flow is taken at first. And then the necessary amount of sample is supplied to the monitoring device via a branch tube directly from the main piping or through a head tank. In this way, the shape of the inlet of a branch tube receiving small rate of flow can be formed to hydraulically make it difficult to clog. Also it is possible to shorten the length of tubing with the small rate flow. However, even though employing such method, it is difficult to maintain a constant flow over a long period of time without maintenance. Therefore, when quantitative accuracy is demanded, it is necessary to conduct an appropriate pretreatment before supplying sample to the measuring equipment.

In case of batch type monitoring device, sample volume is measured at each detection, thus there is almost no problems on the quantitativeness of the sample. Therefore, as far as the sampling unit is concerned, the batch type is easier to operate and maintain.

2.3 PRETREATMENT UNIT

The role of a pretreatment unit differs depending on the substances to be analyzed and detection methods. But generally, the purposes of a pretreating unit are (a) to prevent formation of slime, (b) to dissolve substances contained in suspended solids, (c) to remove or mask interferences, and (d) to adjust samples to best conditions for measurements.

Hazardous substances, particularly heavy metals, in wastewater, are in most cases present in the form of suspended solids. Whatever analytical method is taken, the substances must once be put in a dissolved form. And when the analysis is conducted manually, samples are pretreated by wet oxidation with sulfuric-nitric acids or nitric-perchloric acids to oxidize and decompose substances. However, as it is structually and materially difficult to incorporate such powerful oxidizing process in the automatic monitoring units, pretreatment to merely lower pH of the sample by dosing acid is normally done. When compared to wet oxidation, mere lowering of pH values can not be considered a perfect pretreatment method. But it is effective to a certain degree to dissolve the heavy metals in suspended solids.

Figures 2 ~ 6 show the results of experiments on solubilization of heavy metals in suspended solids by pH adjustments and heating. Samples are raw sewages taken at 4 to 8 sewage treatment plants. These samples were added with a hydrochloric acid for pH adjustment and then filtered. To examine the effect of heating, a part of the sample was heated prior to the filtration. The concentrations of metals in the filtered solutions were analyzed and the dissolved fraction of each metal was calculated. 300°C hot plates were used for heating at about 95°C of water temperature for five minutes. The analysis was conducted by the atomic absorption spectrophotometry (direct aspiration) after wet oxidation of the sample. In case when the metal concentrations in raw sewage was too low to analyze, return sludge was added to the sample to raise the concentrations.

Fig.-2 Solubilization of Heavy Metal in SS in Wastewater by pH Adjustment —Chromium

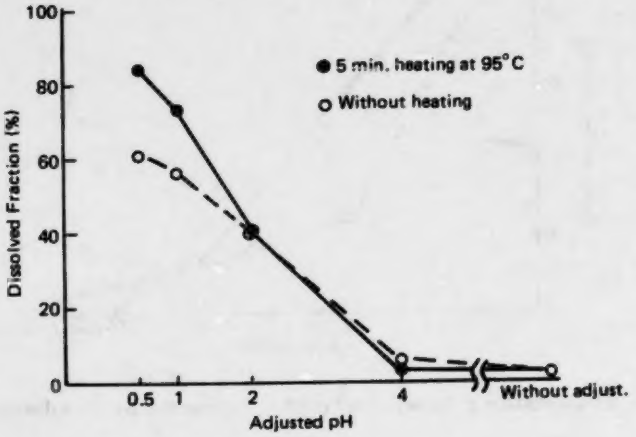


Fig.-3 Solubilization of Heavy Metal in SS in Wastewater by pH Adjustment —Copper

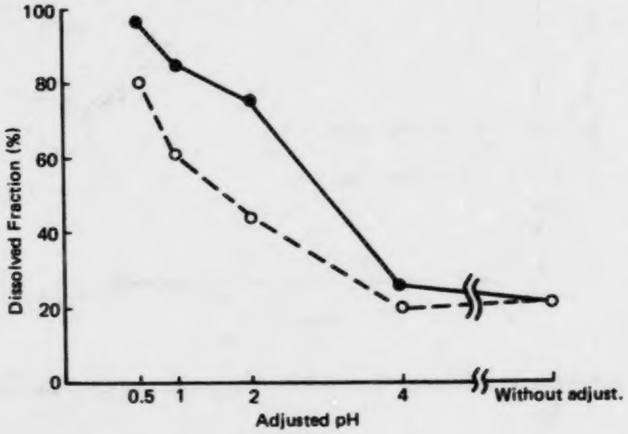


Fig.-4 Solubilization of Heavy Metal in SS in Wastewater by pH Adjustment —Iron

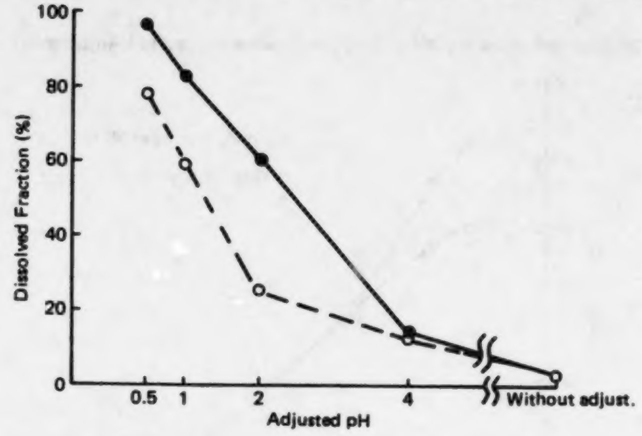


Fig.-5 Solubilization of Heavy Metal in SS in Wastewater by pH Adjustment —Zinc

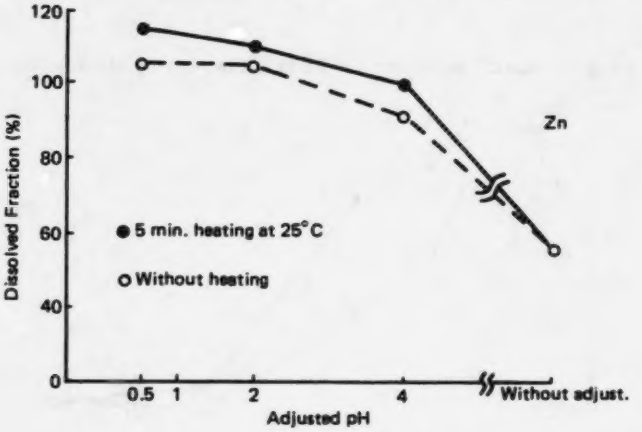
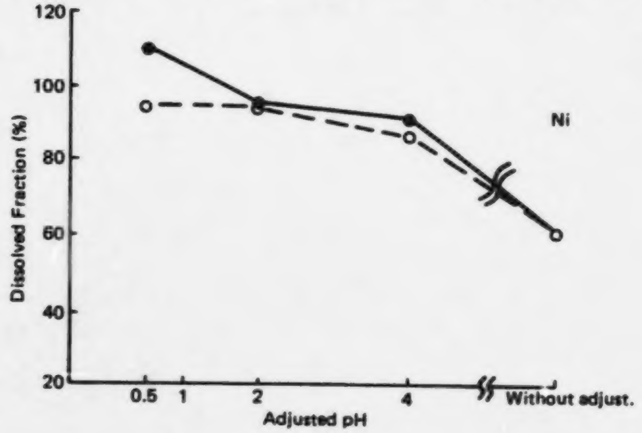


Fig.-6 Solubilization of Heavy Metal in SS in Wastewater by pH Adjustment —Nickel



As understood from the data of the samples without pH adjustment in these figures, a good portion of metals in the sewage was contained in the suspended solids. Especially, dissolved fractions of chromium and iron in the unadjusted samples were quite small. Although the dissolved fractions of zinc and nickel were comparatively high, they were about 50 percent. The lowering of pH is effective to increase the dissolved fraction as shown in the figures. At pH value of 2, zinc and nickel were almost totally dissolved. But with regards chromium, copper and iron, only 80 percent of them were dissolved even when pH value was lowered to 0.5. Heating together with the lowering of pH was found to be effective, and the dissolved fractions of chromium, copper and iron increased to about 90 percent at pH 0.5. Incidentally, the values shown in the figures are average values, and there were deviations of data between 10 and 30 percent expressed in standard deviation. The relationship between pH and dissolved fraction might be dependent on characteristics of wastewater. Therefore, in the process of dissolving substances in suspended solids, measurement errors to some degree would be produced.

Concerning prevention of slime formation, since disinfection is conducted by both acid addition and heating, pretreatment for dissolution of metals can prevent formation of slime.

As to removal of interferences, methods may differ depending on the parameters to be measured and methods of measurement, thus making it difficult to generalize the methods. However, as far as the analysis of heavy metals are concerned, organics are the common interferences. Since it is not wise to automate wet oxidation process for automatic monitors, the activated carbon adsorption may be only process to be acceptable. The adsorption of heavy metals contained in the sample to activated carbon was very small under acidic condition.

2.4 DETECTING UNIT

Measuring methods for automatic water quality monitors for hazardous substances include automated colorimetry, ion selective electrodes and polargraphy. Atomic absorption spectrophotometry, high frequency plasma spectrography and X-ray fluorometry may be included, although they are too sophisticated in mechanism and considered unsuitable for on-site automatic measurement. The possibility of application of each method is listed in Table 1.

Table-1 Analytical Methods and Their Applicability to Automatic Monitor

Substances / Methods	CN	Cd	Pb	As	Org-P	R-Hg	T-Hg	Cr(VI)	T-Cr	Phynols	Cu	Zn	D-Fe	D-Mn	F
Automated Colorimetry	○		△	△				○	○	○	○	○	○	△	△
Ion Selective Electrode	○	○	○								○				○
Polarography	○	○	○								○				
Atomic Absorption		○	○	○		△	○	△	○		○	○	○	○	
Others	High Frequency Plusma Spectrophotometry and X-ray Fluorometry for Metals, and UV Absorption for Cr (VI)														

Notation ○ : Possible, △ : difficult

2.4.1 Automated Colorimetry

The most orthodox detecting method in automatic monitoring is automated colorimetry. When strict quantitative analysis is required, pretreatment is indispensable. However, for actual application, it is considered difficult to develop a monitoring device having complicated functions, such as wet oxidations, solvent extraction and so on. Therefore, the analytical methods for monitors are limited to those with relatively simple procedure, yet giving reasonably accurate results.

The following methods are some examples: Bathocuproin method for cupper, diphenylcarbazide method for chromium, 4-aminoantipyrine method for phenol, isonicotinic acid-pyrazolone method of cyanide, etc. Even with these methods it is necessary to perform pretreatment, such as oxidation of trivalent chromium for analysis of total chromium, reduction of trivalent iron for measurement of total iron, reduction of divalent copper for appraisement of total copper, and distillation of cyanide for valuation of total cyanide.

When employing automated colorimetry, the problems involved are troubles caused by using wet chemicals and danger of resulting large measurement errors due to lack of complete pretreatment procedure to eliminate interferences. However, the latter is a common problem to any measurement method.

Since it is difficult to totally remove suspended solids, the device must have a mechanism that can correct the sample blank. Although sample blank correction can be done by the double beam system and others for continuous type monitors, the batch measurement is easier in this respect.

2.4.2 Ion Selective Electrodes

Ion selective electrodes available on the market are those for cadmium, lead, copper, cyanide and fluoride measurements. Fluoride electrode has high selectivity and less interferences. However, as regards other electrodes, they are many and serious interferences. The well known ones are Hg^{2+}, Ag^{+}, Fe^{3+} and S^{2-} for cadmium, lead and copper electrodes, and S^{2-} for cyanide electrode. It is not that difficult to insert in the unit a process of dosing masking agents or other processes to eliminate the interferences by these inorganic ions. The greatest problem is the interferences caused by organics present in the wastewater. Table 2 shows the selectivity coefficients of selected organic substances. The Table shows that ionized organics cause extremely large interference. Actually, when measurements are done without removing the organics, for example using cadmium electrode, the measured values sometimes are lower by one order than the true concentration. Accordingly, when employing ion electrodes for the detecting unit, it is imperative to remove organics by activated carbon and soon.

Table-2 Selectivity Coefficients against Selected Organic Substances

Electrodes Substances	Cadmium	Copper	Lead
Ethyl Alcohol	$10^5 <$	$10^5 <$	$10^4 <$
Glucose	$10^4 <$	$10^4 <$	$10^4 <$
Sucrose	$10^5 <$	$10^5 <$	$10^4 <$
Acetic Acid	2.1	0.6	1.2
Succinic Acid	1.9	28	7.2
Tartaric Acid	21×10^2	84	54
Cifric Acid	2.2×10^3	57	2.1
Linoleic Acid	$10^4 <$	$10^4 <$	$10^4 <$
Oxalic Acid	0.27	1.1	2.8
L-Ascorbic Acid	4.8	64	21
L-Gultamic Acid	2.2	$10^3 <$	4.0
Casein	3.1×10^2	8.4	54
Peptone	2.4	4.9×10^{-2}	15
Dextrin	1.6×10^2	6.1	2.3×10^2
Powdered Cream	18	2.1×10^3	34
Urea	7.0×10^4	1.8×10^2	90

Note : Figures in the table are the averages of the selectivity coefficients at dadmium concentrations of 0.01, 0.1 and 1.0 mg/l for the Cd electrode, at copper concentrations of 0.3, 3.0 and 30 mg/1 for the Cu electrode, and at lead concentrations of 0.1, 1.0 and 10 mg/1 for Pb electrode, respectively.

2.4.3 Polargraphy

Ordinary polargraph with dropping mercury electrode is occasionally used in the automatic monitors for rivers. But owing to the difficulty in coping with interferences such as organics, reducing substances, iron and other interfering substances, and the difficulty in maintaining proper condition of the dropping mercury electrodes, ordinary polargraph are considered unsuitable for monitors for wastewater. However, as the anode stripping voltimetry, which is one type of polargraphy, allow the use of mercury electrodeposited graphite electrode or glassy carbon electrode, it has a possibility for being applied in automatic monitoring devices.

There are three heavy metals possible for measurements at the same time by the anode stripping voltimetry – cadmium, lead and copper. According to a test employing a mercury electrodeposited graphite electrode, it was possible to obtain comparatively high precision measurements of lead and copper by dissolving these metals by acid addition followed by filtration and removal of dissolved oxygen. Organics contained in wastewater did not cause strong interference. However, with regards to cadmium measurements, when measurements were conducted in a sample pretreated in the above mentioned way, the characteristics of the electrode deteriorated tremendously thus the removal of organics was considered necessary.

The anode stripping voltimetry has given rise to such problems as expensive maintenance cost due to electrodeposition of electrodes at each 100 or 200 times of measurement and complicated data processing due to the necessity of obtain intergal values instead of peak volues of output electric current.

2.4.4 Atomic Absorption Spectrophotometry

The atomic absorption spectrophotometry is the most popular method in conducting manual analysis of heavy metals in wastewater. And the development of automatic monitors with this method as detecting unit is advancing to a certain stage. Especially with regards to mercury analysis, the flameless atomic absorption spectrophotometry was regarded as the only practical measurement method.

When measurements are conducted by the cold vapor atomic absorption method, the analysis of inorganic mercury is said to be effectively done by performing pretreatment of dosing ammonium chloride and heating. It is also reported that the measurement of total mercury was effectively conducted only by the oxidation with potassium permanganate under acidic condition.

Concerning other heavy metals, when the flame atomic absorption spectrophotometry and the flameless atomic absorption spectrophotometry are compared, it is considered that the flameless method is more appropriate from the view points of removing interferences by organics and the possibility of on-site installation. However, in case of the flameless method, it is difficult to automate the sample injection to the carbon atomizer. At this stage, atomic absorption spectrophotometry is considered to be too sophisticated and complicated for monitors being installed on-site.

2.4.5 Others

When ignoring complexity of the equipments, difficulty in operation and maintenance, and economy, the high frequency plasma spectrography and the X-ray fluorometry would be good methods since they have less interferences than other methods and have the merits of measuring various elements at the same time. However, the possibility of putting these methods to practical use at the present stage is dim.

Hexavalent chromium can be measured by ultraviolet absorption. This method utilizes the property of hexavalent chromium in alkaline solution that exhibits ultraviolet absorption at about 370 nm wave length, while trivalent chromium does not adsorb UV in the area of this wave length. The measurement of the hexavalent chromium concentration is seeked from the difference in the absorbance before and after the hexavalent chromium is reduced to trivalent in the sample. The greatest intereference is ferric ion. When the ferric concentration is high, the it must be removed.

3. CURRENT STATUS OF DEVELOPMENT

On consignment from the Ministry of Construction, the Association of Electrical Engineering has organized a committee composed of users and manufacturers within its framework and has so far tried to develop and improve automatic monitoring devices for total cyanide, total chromium, hexavalent chromium, cadmium and copper. In performing these works, specifications of the devices, such as accuracy of measurement, which are considered achievable, are determined, and long term field tests are to be conducted to confirm that the specification are met.

Of automatic monitoring devices, the total cyanide monitor, for the time being, has been developed, and its standard specifications were already published. However, concerning others, they are presently under development and are slated for commencement of field tests by the end of this year.

3.1 TOTAL CYANIDE MONITOR

In the development of total cyanide monitor, the most important thing is to remove interference by sulfur ion and others. As a result of laboratory and field tests on the methods of pretreatment, it has been decided to adopt the method to oxidate sulfur ion by potassium permanganate under acidic condition prior to distillation and the method to remove sulfur ion by granular lead peroxide after distillation. The measurement of cyanide is conducted by the cyanide electrode.

The field tests were conducted for about two years at an ordinary municipal sewage treatment plant and at a treatment plant handling industrial wastewater mainly consist of metal finishing wastes. And improvements of the devices were made during the field test. It was found that some kinds of organics could be transferred into the distillate during distillation and could interfere the measurement. Therefore, activated carbon column is necessary to remove organics in the distillate when occasions demand. Through these tests, it was confirmed that the accuracy of measurement was between the range of –0.15 pCN and 0.25 pCN provided that there were no unforeseen interferences contained in the sample. The larger error on the positive side was due to the fact that the influence of interferences acted more strongly on the 'plus' side.

There are three types of monitoring devices designated by the standard specifications: one batch type and two continuous types. Concerning the details, refer to the "Standard Specifications of Automatic Total Cyanide Monitoring Devices" issued by the Association of Electrical Engineering in March 1978.

3.2 TOTAL CHROMIUM MONITOR

A few types of automatic total chromium monitors based on the diphenylcarbazide method are already being put on the market. However, it is doubtful whether these monitors can be used for the measurement of chromium in wastewater which contains organic reducing substances in high concentration. Therefore, based on the hitherto laboratory tests, a series of field tests to improve and modify the monitoring devices presently marketed will begin from late this year. The major purposes of the field tests are to study the effects of preliminary aeration prior to addition of potassium permanganate and the possibilities of using other oxidizing agents such as sodium hypobromite instead of potassium permanganate. Based on the results of these field tests, standard specifications are to be made.

3.3 HEXAVALENT CHROMIUM MONITOR

With regards to the measurement of hexavalent chromium, a few types of automatic monitoring devices employing the diphenylcarbazide method are also on the market. But when these monitors are used for the measurement of Cr (VI) in the sewage, sample blank becomes extremely large and it often exceeds the range of automatic correction. Furthermore, direct measurement of Cr (VI) by the diphenylcarbazide method in samples containing a large amount of reducing substances may be doubtful, thus gives rise to studies of introducing the aforementioned ultraviolet adsorption method in the monitoring devices. However, the ultraviolet adsorption method, too, faces interference by iron. Ferric hydroxide formed in the alkaline solution absorbs ultraviolet, and 10 mg/l of ferric ion gives the interference equivalent to about 1.5 m/l of chromium. Therefore, when this method is applied to water sample containing a large amount of ferric ion it must first be removed.

3.4 CADMIUM MONITOR

Concerning cadmium measurement, as there is no appropriate colorimetric analyzing method for automatic monitoring, studies are presently being advanced to develop an automatic monitoring device with a detecting unit using ion selective electrodes. However, as mentioned previously, cadmium electrode has an increasing large number of interferences. According to the hitherto basic experiments, results show that interferences by organic were removed to some extent by passing the sample through activated carbon column under acidic conditions and that interferences by inorganic ions were eliminated by addition of proper masking agents. Masking agents to remove metal ions, for example, are phosphates to remove Pb^{2+}, Ag^{2+}, Fe^{3+}, etc.; L-ascorbic acid as against Cu^{2+} and oxidizing substances; and sodium tetraphenyl borate to cope with Ag^{2+} and Cu^{2+}. For others, by utilizing the nature of cadmium that it is adsorbed to anion exchange resin under acidic condition with hydrochloric acid, extraction of cadmium from the sample is being studied.

With regards to studies of cadmium monitoring devices, field tests are slated to commence at the end of this year to conduct comparasions of effects of the above mentioned pretreatment methods and to decide on the apt mechanical structures of the devices considered most appropriate for practical application.

3.5 COPPER MONITORING UNIT

The bathocuprion method with extremely few interferences is a colorimetric analyzing method of copper and serves as an excellent detecting method for an automatic monitor. Concerning copper monitoring units employing the bathocuprion method, field tests are expected to begin from the latter part of this year. And surveys will be conducted on measurement precision, operation and maintenance of the devices.

3.6 OTHERS

Besides cooperating in developmental studies of various monitoring devices on consignment by the Ministry of Construction, manufacturers have their own development projects. As mentioned before, mercury monitoring devices employing the frameless atomic absorption spectrophotometry are presently marketed, and also a monitoring device for heavy metals having flame atomic absorption spectrometry detecting units is on the market. However, as these devices are not always designed for application to wastewater that contains a large number of interferences, there are many problems involved when they are to be used for wastewater monitoring.

Presently, automation of X-ray fluorometry and high frequency plasma spectrophotometry has advanced to a stage of developing laboratory type automatic instruments. And, depending on the requirement of the users, there is a possibility of these methods being adopted as detecting units in automatic monitoring devices in the future.

4. CONCLUSION

The current status of the development of automatic water quality monitoring devices for hazardous substances in wastewater and its related problems have been discussed in this report. At the present stage, the only reliable monitoring unit that has been completed is that for cyanide measurement. As regards chromium, cadmium and copper, the outlook for developing monitors for these substances is bright.

The strenous efforts until now has been exerted to develop monitors to give as accurate data as possible in practical sense. However, generally speaking, as the measurement accuracy requires complicated mechanism, the accuracy required and the cost as well as difficulty in operation and maintenance are in a counteracting relation. Therefore, studies should be made on the 'software' of the monitoring system, such as the role of automatic monitors in the total monitoring system and cost benefit relationship. And the results of these studies should be forwarded as a feed-back information to set the targets to develop automatic monitoring devices.

II. PERFORMANCE OF CONTINUOUS WATER QUALITY MONITORING EQUIPMENTS FOR SEWAGE TREATMENT

INTRODUCTION

Since 1972, the Public Works Research Institute of the Ministry of Construction has conducted a series of research activities on advanced waste treatment using a pilot plant constructed at the existing sewage treatment plant in three cities, Shitamachi plant in Yokosuka, Toba plant in Kyoto and Nishiyama plant in Nagoya, with full collaboration from respective municipalities. These researches also cover the applicability test on the continuous water quality monitoring equipments available on the market which are indispensable for effective operation and control of the advanced waste treatment at the said plants.

The evaluation of continuous water quality monitoring equipments is tested mainly at Kyoto plant and Nagoya plant. Kyoto plant, which is intended to treat secondary effluent, is provided with the functions essential for that specific purpose such as chemical precipitation, sand filtration, activated carbon adsorption, and break point chlorination. Nagoya plant, on the other hand, is an actual plant designed to upgrade the functional aspects of the existing plant by adding coagulant, and it also has sand filters for experimental purposes.

This paper introduces some of the monitoring equipments installed at the two plants which are generally applicable for advanced waste treatment, with specific reference to their performance and problems in their actual operation.

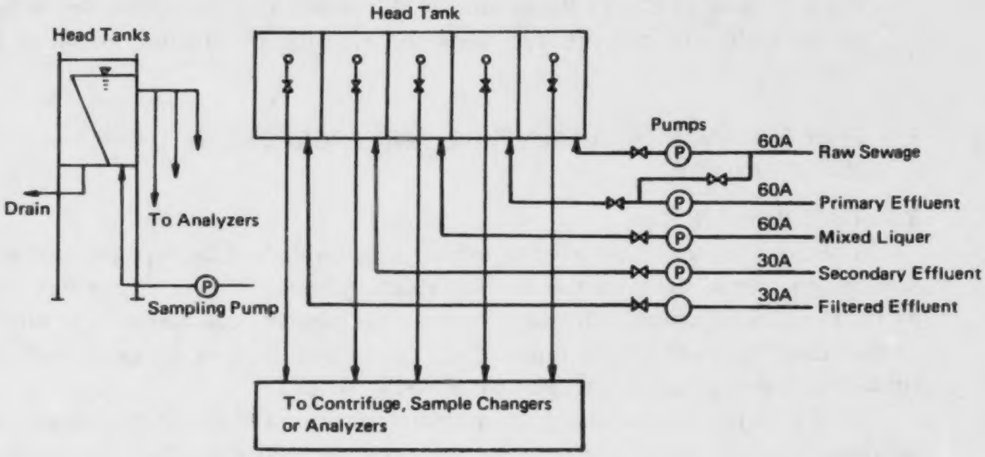
1. SAMPLING PROCEDURE

In automated water quality analysis of sewage, the sampling procedure or the method of leading sample sewage to the terminal water quality monitoring equipments (hereinafter referred to as the "water quality analyzers" or "analyzers") bears very closely upon smooth and accurate measurement. This is due to the inherent characteristics of sewage, especially raw sewage, which contains lots of solids and organic matters. The solids are prone to be accumulated in the pipes and the organic matters are liable to cause slime formation, so that the piping system is easily clogged up and changes in water quality are liable to occur. Particularly in the case of raw sewage which contains lots of coarse solids, sampling must be carried out constantly to maintain sample continuity and the whole configuration of the sampling system must be designed with care taken in the selection of sampling pumps and valves.

Kyoto plant is a tertiary plant intended mainly to treat secondary effluent, so that its sampling system is less influenced by the above mentioned characteristics of sewage than at secondary treatment plants. In addition, as the conveyance distance is short, it is not confronted with any specific difficulty in leading sample from each specified sampling point to the analyzer by making use of the head in the conveyance route or by means of exclusive sampling pumps.

In the case of Nagoya plant, however, the applicability test was conducted by making use of the actual plant, so that it was necessary to drive all effluents, ranging from raw sewage to final effluent, to the shed designed exclusively for water quality analysis. For this reason, raw sewage and effluents were pumped up continuously from each sampling point to specially dsesigned head tanks, and thence to respective analyzers through pipelines with a suitable head. Further, a centrifuge (8,000 rpm) run by an industrial motor was installed immediately beneath the head tank to send the sample free from suspended solids to those analyzers whose performance could be degraded by the inflow of sample containing the solids. A sample changer with an electromagnetic valve was also used for measurement of a plural number of samples by a single analyzer. The flow chart of sampling at Nagoya plant is shown in Fig. 1.

Fig.-1 Flow Chart of Sampling (Nagoya)



By reason of their functional and structural characteristics, pH meter and sludge density meter were installed at each measuring point outside the sampling system described above.

The sampling pumps were of self-contained volute type, with a suction port diameter of 60 A for raw sewage and primary effluent and 30 A for other effluents, and the flow form them to the head tanks was adjusted by means of delivery regulating valves to maintain an optimal rate of overflow from the head tanks.

After about two and a half years' operation of the above sampling system, the following problems were brought to the fore.

1) Scums were formed in the head tanks, and their admixture in the sample caused dispersion of measured values. This drawback could be coped with by cleaning the head tank every morning for most effluents but not for raw sewage which contained lots of solids. Thus, the system proved to insufficient for sampling raw sewage.

2) A diversity of troubles were encountered in the pumping of raw sewage. To cite an example, the strainer of ordinary suction pipe became clogged up rapidly due to the inflow of paper, debirs and dirts contained in raw sewage, often causing overload of the pump. By introducing a large stainless steel basket type strainer (400 x 400 x 550 mm, 5 mm square), this trouble was mitigated to a substantial extent, but it was still necessary to carry out screening once a week or so.

3) As the raw sewage sampling pipeline installed at Nagoya plant was long (approx. 70 m), it was subjected to frequent clogging, causing gradual decline of the pump delivery rate. Four solution of this problem, the delivery side piping of primary effluent pump was connected with the said raw sewage sampling pipeline to flush is at suitable intervals with the primary effluent as backwash water.

4) Pretreatment by the centrifuge for removal of solids had to be conducted every 5 to 7 days to remove concentrated solids caught inside the rotor. Neglect of this pretreatment caused the decline of SS removal rate, which resulted in the decomposition of concentrated solids and emission of an offensive odour in summer.

2. PERFORMANCE OF ANALYZERS, AND PROBLEMS IN THEIR MAINTENANCE

2.1 TURBIDIMETER

The turbidimeter used for water quality monitoring is of the type designed for measuring scattered lights on the free water surface formed on top of the flow cell by the overflow of sample effluent. This meter is generally considered to be suited to the measurement of sewage turbidity for two reasons: it does not come in direct contact with sewage, and sample volume is usually abundant.

At Kyoto plant, 10 units of this turbidimeter are installed for the purpose of continuous water quality monitoring of secondary effluent, chemical clarifier effluent. At Nagoya plant, 6 units are installed for turbidity measurement of raw sewage, primary effluent, secondary effluent, and filltered effluent. All the 16 units are operated at a high rate of nearly 100%.

As turbidity is not the only or a sufficiently credible index in sewage treatment, it is important to clearify the turbidity vs. SS relationships which are shown in Figs. 2 ~ 4 for the two plants. Fig. 2 shows the relationships at Kyoto plant between the turbidity of secondary and filtered effluents and their SS concentration obtained by manual analysis. Fig. 3 also shows the turbidity-SS relationships between effluent. As seen in the two figures, secondary and subsequent effluents present a marked correlation between turbidity and SS concentration, indicating that SS concentration can be estimated by turbidity measurement. Fig. 4 shows the turbidity-SS relationships between raw sewage and primary effluent at Nagoya plant. This figure indicates that raw sewage does not present any notable correlation between turbidty and SS concentration, suggesting that it cannot be justified to represent SS by turbidity. This is assigable to the working principles of the turbidimeter which does detect settlable solids of relatively large size.

Fig.-2 Turbidity ~ SS (Secondary Effluent and Its Filter Effluent)

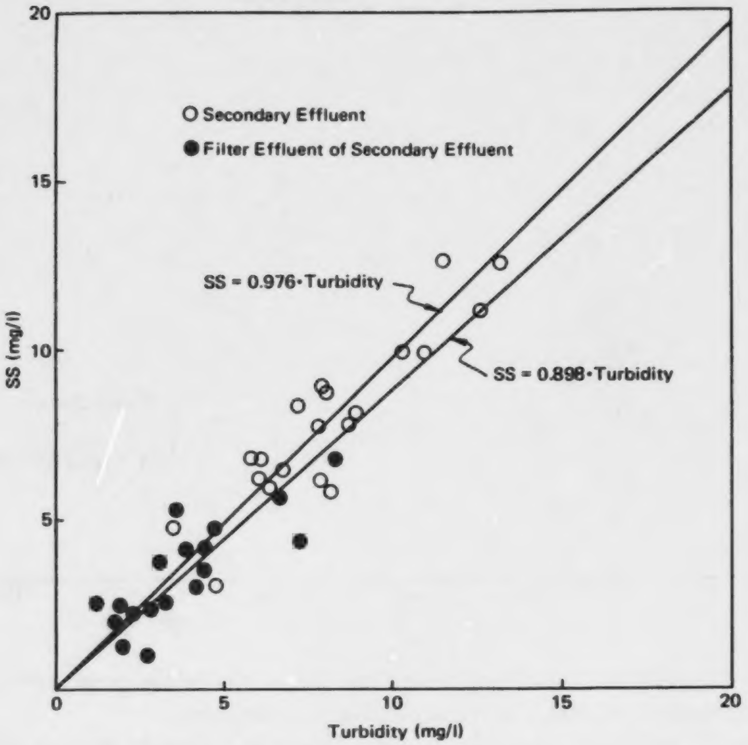


Fig.-3 Turbidity ~ SS (Alum Precipitation Effluent and Its Filter Effluent

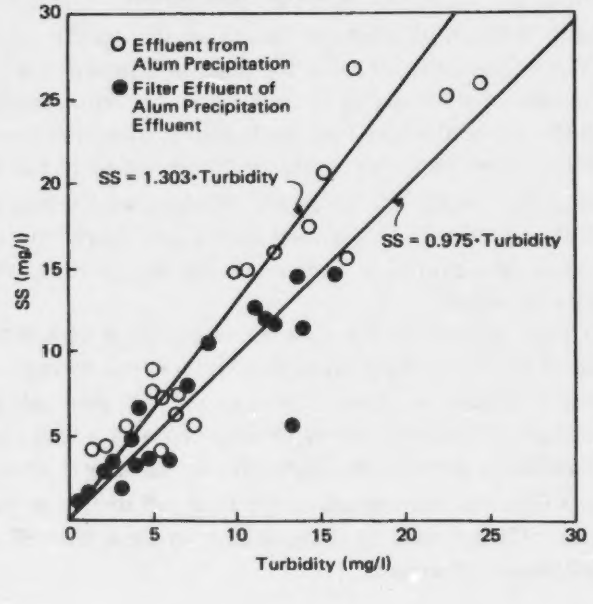
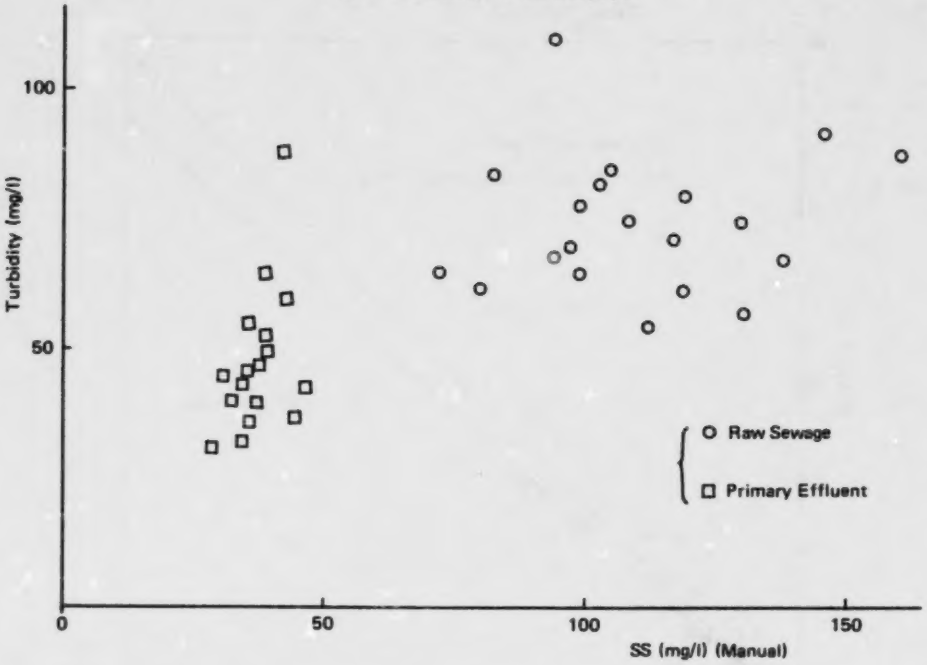


Fig.-4 Turbidity vs. SS (Nagoya)



From the experience obtained at the two plants, the follow proposition can be advanced in connection with continuous turbidity monitoring.

1) Secondary effluent, chemical clarifier effluent and filtered effluent present a comparablly high correlation between turbidity and SS concentration. In so far as these effluents are concerned, therefore, it is feasible to obtain SS concentration from the measured values of turbity with a fairly high accuracy. However, as the relationships betwen these two factors vary according to sample quality, it is necessary to establish the relationships for each effluents.
2) As for raw sewage, the validity of estimating SS concentration from measured turbidity values is questionable. At both plants, turbidimeters were used for quality monitoring of backwash waste of filters which had a higher SS concentration than raw sewage. In the case of this backwash waste, the correlation was rather high, which may be ascribed to the fact that activated sludge flocs which are relatively uniform in size constitute the greater part of suspended solids of backwash waste.
3) The effluent flow velocity on the surface of flow cell is limited to an extent by reason of the system's working principles. Hence, raw sewage containing lots of coarse solids formed a debris film covering the flow cell surface, which caused dispersion of measured values. Formation of this film was so rapid that it was not possible to prevent the dispersion by cleaning it away once a day or so. The debris film was also formed on the flow cell surface of primary effluent and secondary effluent, making it imperative to clean the cell at least once a day for stabilized measurement.

4) Certain minor differences in measured values were observed between respective turbidimeters, which are likely to have been noted among other analyzers. Specifically, there were cases where the turbidity of the same sample differed from turbidimeter to turbidimeter. This is because even if a number of meters are calibrated all at the same time, the measured value inevitably involves a deviation from the correct value varying in degree for each meter with the lapse of time. This fact must be taken into careful consideration when measured values of low turbidity effluents such as filtered effluent and carbon effluent are to be sued for absolute turbity comparison between such effluents. Never-the-less, this trouble can be averted by measuring the turbidity of a number of samples with the same turbidimeter by means of the aforementioned sample changer.

2.2 UV (ULTRA VIOLET) PHOTOMETER

The abosrbance in the neighbourhood of extinction of 254 mm beam in water contanining organic matters in known to have a certain correlation with the organic matter concentration measured in terms of COD and BOD. In general, UV photometer is basically designed for measuring absorbance through a flow cell, and it is simple in structure and easy for maintenance. Where the organic matter concentration in sewage of treated effluent can be represented by UV absorbance, its measurement can be greatly facilitated by the use of UV photometer.

The UV photometers installed at the two plants are capable for measuring the absorbance of an extinction of 254 mm beam using a mercuric lamp, and can effect correction to the influences of turbidity of taking the ratio of transmitted light intensities of ultraviolet rays and visible rays.

At Kyoto plant, 2 units of UV photometers are combined with the sample changer for continuous quality monitoring of secondary effluent, chemical clarifer effluent, filter effluent and carbon effluent. At Nagoya plant, on the other hand, 1 unit is installed for combination with the sample changer for measurement of primary effluent, secondary effluent, and filtered effluent.

Although a period of about one and a half years has elapsed since the commencement of the measurement, a very high operational rate is exhibited by all the three photometers. In the earlier stage of measurement, however, the following problems were experienced.

1) Measured values showed fluctuations due to temperature changes in the neighbourhood of the flow cell.
2) Ozone was generated near the mercuric lamp, emitting its peculiar smell.

The former trouble was solved by improving the measuring unit so as to maintain a constant temperature (40° C), and the latter by inserting activated carbon in the measuring unit to adsorb ozone.

Fig. 5 shows the realtionships between COD (permanganate) and UV absorbance as obtained by measurement conducted for about half a year (December to June) at Kyoto plant on effluents at different stages of treatment. The figure presents the general tendency that both COD and UV absorbance decline with the progress of sewage treatment from secondary effluent to chemical clarifier effluent, and further to carbon effluent, but indicates the difficulty in estimating COD from UV absorbance.

Fig.-5 UV vs. COD

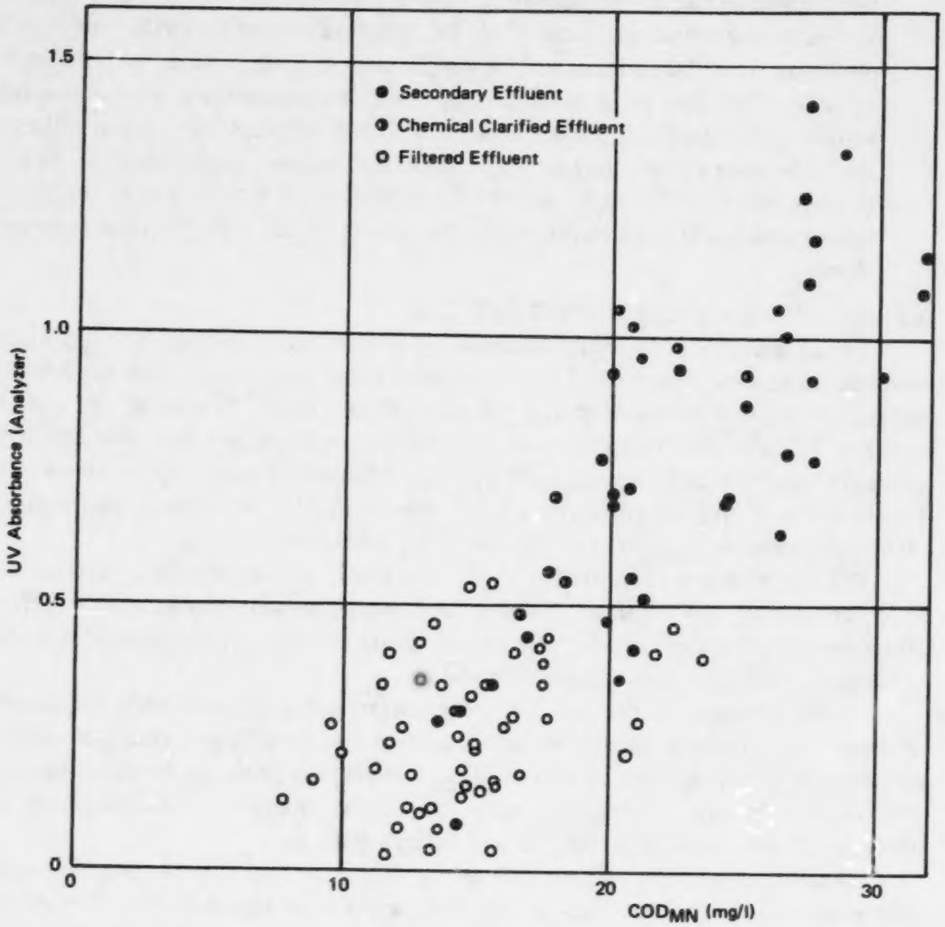


Fig. 6, which illustrates the COD vs. UV absorbance relationships as obtained by measurement in a period differing from that mentioned above, indicates that COD and UV absorbance presenta marked correlation with each other if the survey period is the same, but the correlation changes if the survey period is not the same.

Fig. 7 shows the relationships between the absorbance measured by the UV photometer at Nagoya plant and TOC (value measured by continuous TOC meter). The figure indicates that UV absorbance vs. TOC relationships vary according to the treatment stage just as the abosrbance-COD relationships mentioned above, and also makes it clear that when TOC is higher, its value is vulnerable tó a large fluctuation even by a slight difference in UV absorbance. It seems that there exists no clear correlation between the turbidity measured by turbidimeter and the UV absorbance, and that turbidity correction was effected without any hindrance.

Fig.-6 UV vs COD (Seasonal Change)

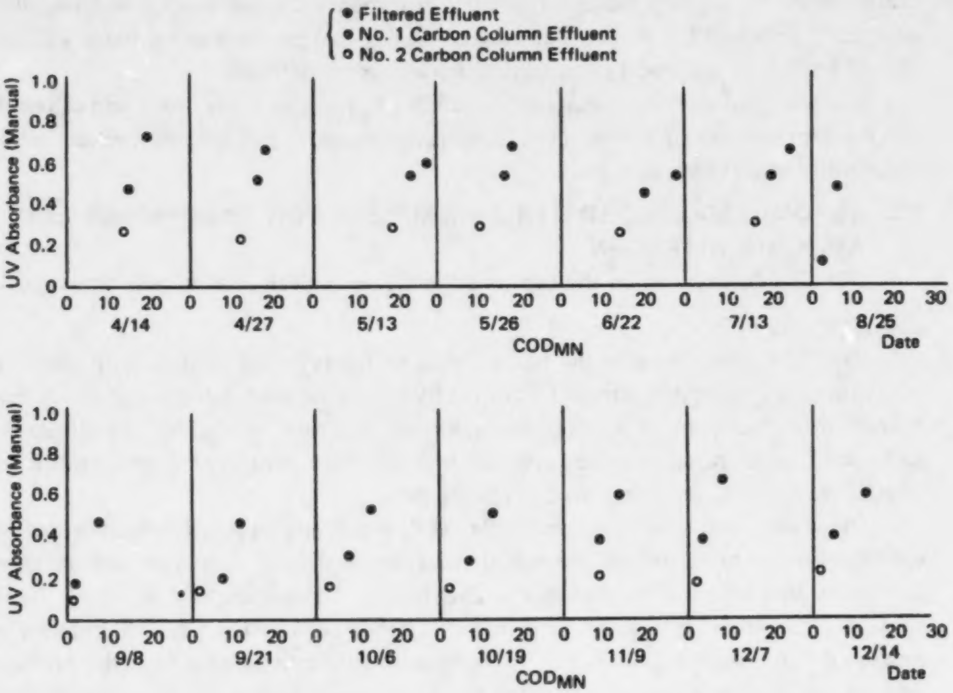
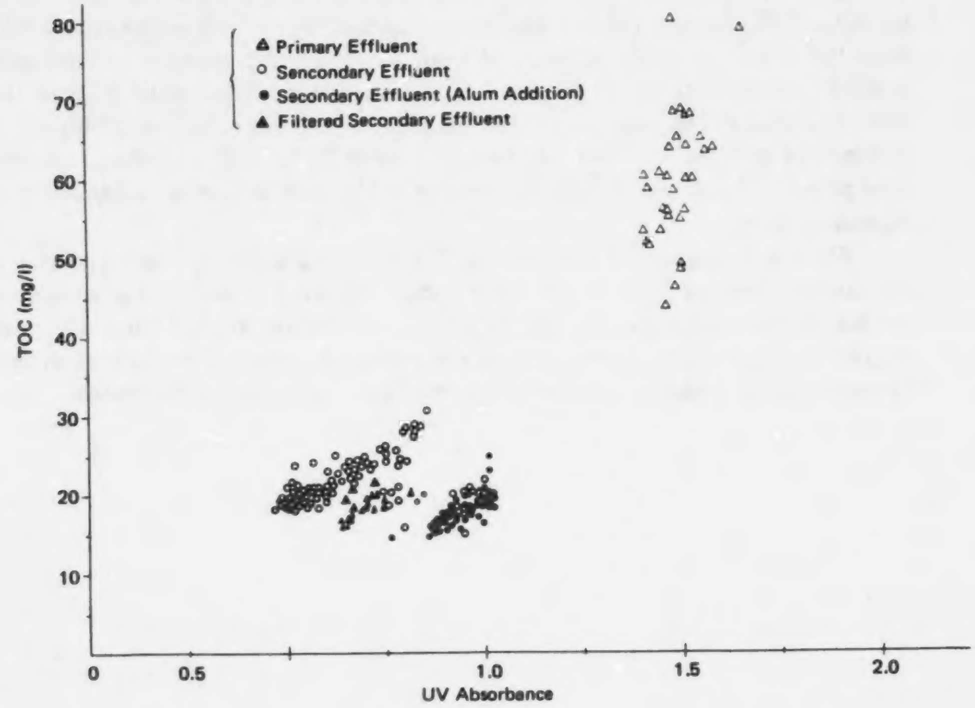


Fig.-7 Relationship between UV Absorbance and TOC (Automatic Analyzer)



From the results introduced above, it appears questionable to obtain the absolute values of organic matters from UV absorbance by continuous water quality analyzers, although the abosrbance may be used in setting the control point within a certain limit, e.g., the point of breakthrough of carbon effluent.

As described already, maintenance of UV photometer is very easy and calls only for the replacement of the mercuric lamp (one thousand and several hundred hours) and routine inspection services.

2.3 AUTOMATED COLORIMETRIC ANALYZER FOR PHOSPHORUS AND AMMONIA NITROGEN

Detailed description on the captioned analyzer is given elsewhere in this report.

2.4 TOC METER

The TOC meter used at the two plants is of the type designed to drop pH value of continuously supplied sample by adding hydrochloric acid, purge inorganic carbon therefrom by bubbling in an exclusive scrubber, and then obtain TOC by measuring CO_2 with an infrared analyzer after incinerating the sample at a temperature of about 800°C. CO_2-free air is used as carrier gas.

By reason of its working principle, TOC meter presupposes instantaneous incineration of sample, so that the sample must be very small in volume and the pipeline must also be small in diameter. This makes it inevitable for the meter to be subjected to strong influences of suspended solids such as the large fluctuation of measured value resulting from the infiltration of suspended soilds into the furnace, and the deflection of measured value of total TOC which is caused when part of suspended solids in introducted in the pipeline running from the sampling point through the analyzer. Fig.8 is a comparison between TOC values of primary effluent, secondary effluent and filtered effluent as obtained by manual analysis (on samples collected from the outflow point of head tank) and the readings of continuous analyzer recorded at the same time. In the manual analysis, values of both total TOC and soluble TOC were obtained. As seen in this figure, the values obtained by continuous analyzer are close to those of soluble TOC, suggesting that a considerably portion of suspended solids is removed in the sampling line downstream of the measuring point.

Fig. 9 is a comparison between the TOC values obtained by manual analysis on the sample collected from the scrubber and the readings of continuous analyser recorded at the same time. As can be seen in the figure, the two show a fair conformity to each other, indicating that the measured values of continuous analyzer become stabilized after suspended solids have been removed to some extent.

Fig.-8 Comparison of TOC, Manual and Automatic Analyses

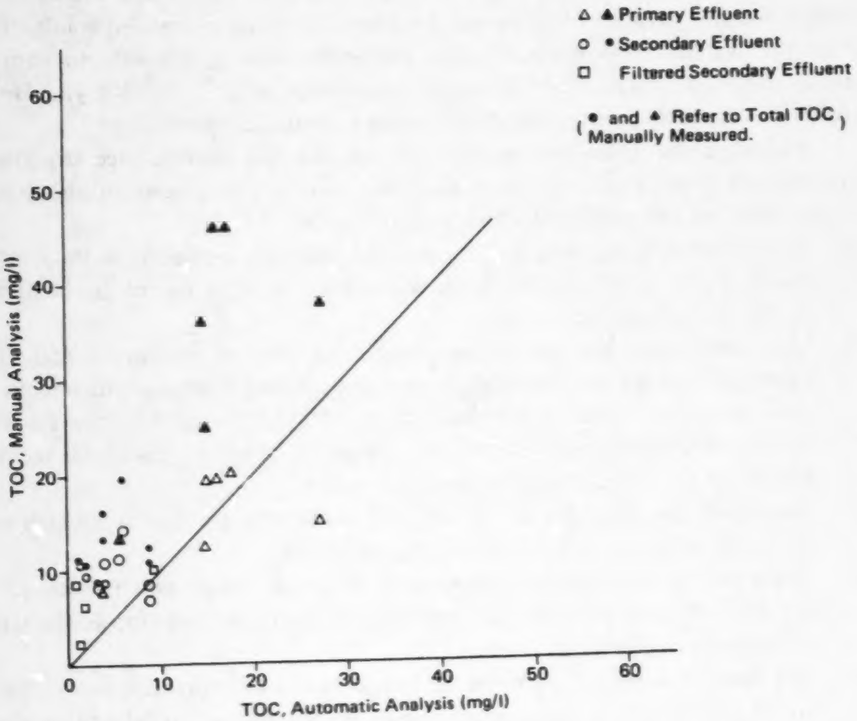
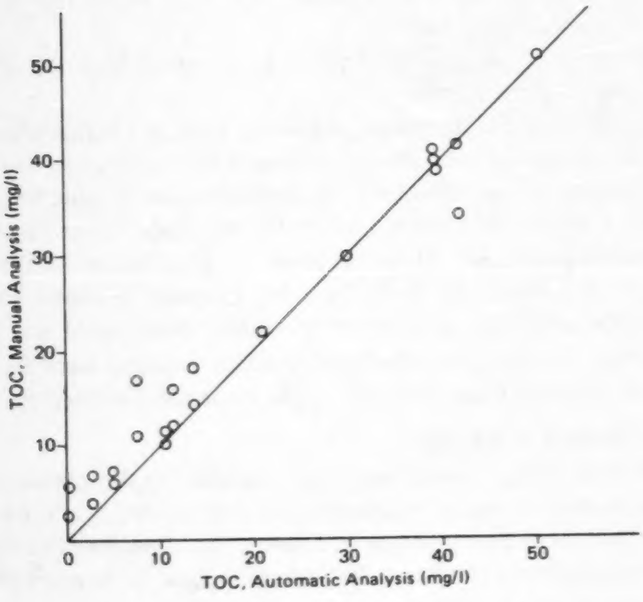


Fig.-9 Comparison of TOC, Manual and Automatic Analyses
— Samples taken at the Outlet of the Scrubber in the TOC Monitor —



Prior to its actural application, the TOC meter mentioned above was considered to be superior to all other continuous TOC analayzers currently available on the market for the purpose of analyzing samples containing suspended solids. Judging from the test data, however, it does not seem fully appplicable to continuous measurement of total TOC including suspended solids. At Nagoya plant, this problem is coped with by pretreatment using a continuous centrifuge.

Although the following operational troubles and maintenance troubles were encountered in the past, none were so serious as to invite suspension for a long time and the TOC meters exhibited a high operating rate.

1) Pipelines developed frequent thermal deterioration especially in the neighbourhood of cell of infrared analyzer where the pipe replacement can be done only by the manufacturer.
2) The carrier gas line was often clogged up due to moisture. Although an electronic cooler was installed behind the cooling condenser to remove moisture, water collected in the dust filter and often clogged up the carrier line This trouble was coped with by modifying the shape of dust filter so as to be able resist the collection of water to an extent.
3) Dehumidifying silica gel had to be replaced frequently due to the high relative humidity of ambient room air used as carrier gas.
4) The scrubber for removing inorganic carbon has a shape which makes its cleaning difficult, and the pipeline supplying sample from the pump to the scrubber is subjected to frequent clogging.
5) The furnace needs to be replaced in less than two years due to its corrosion, and the breaking of heater wires must also be assumed to take place after the lapse of just about the same period.

2.5 pH METER

The pH meters used are of dual sensor type with augmented internal liquid volume (total length : approx. 2 m).

At Nagoya plant, 5 pH meters of this type are installed in the aeration tanks and primary effluent delivery lines.

By calibrating these pH meters once a week, measured values are stabilized at present. The sensor surface becomes soiled considerably in a single day if the meter is installed directly in the aeration tank, but excessive soiling could be prevented by cleaning it once a week and it did not affect the measured values. By reason of its functional characteristics, the pH meter needs an amplifier to be fitted near the extreme end of its sensor. As there were no adequate measures to reduce the humidity near the amplifier, satisfactory measured values could not be obtained from some meters. To cope with the high humidity resulting from the tanks being installed indoors, silica gel filled in a box is replaced at suitable intervals.

2.6 SLUDGE DENSITY METER

Ultrasonic type sludge density meters are installed in designated places in the pipline at both plants; for density measurement of primary sludge and waste activated sludge at Nagoya plant and alum sludge at Kyoto plant. The density meter itself is a virtually maintenance-free instrument, but attention must be directed to the following points in order to obtain accurate measured values.

1) As the declining rate of ultrasonic wave varies according to the changes in sludge properties resulting from the development of bulking and addition of coagulant, it is necessary to carry out calibration frequently. In the density measurement of waste activated sludge conducted at Nagoya plant, it was discovered that 20 dB decline at FS 3% when SVI stood around 100 in summer had to be adjusted to 33 dB decline at FS 3% in winter when SVI rose to about 200 (pipe diameter: 150 mm).

2) The declining rate is high especially for primary sludge. The measured density of primary sludge once showed a value about 1.5 times as large as that obtained by manual analysis even near the uppermost limit of adjustable range (FS – 8%, 45 db decline) (pipe diameter : 300 mm). As primary sludge is subject to little fluctuation of properties and consequently presents a high density correlation between manual analysis and density meter measurement, a predetermined coefficient is adopted at Nagoya plant to obtain the primary sludge density.

3) As the density meter gives the most reliable value around the centre of the measuring range, it is not advisable to set the measuring scale extending unnecessarily far beyond the density range of measured sludge. This caution needs to be observed particularly for waste activated sludge as its density changes largely with the lapse of time.

3. CONCLUSION

In the foregoing pages, the continuous water quality monitoring analyzers installed at Kyoto plant and Nagoya plant have been introduced in outline, together with the problems entailed in their maintenance as disclosed from the record of their past operation.

All the analyzers selected for elucidation in this report exhibited a satisfactory operating rate. Considered from the viewpoint of reliability, however, they can be divided into two groups, those producing fairly stabilized measured values and those liable to produce misleading values unless operated with full understanding of functional and structural characteristics. In terms of maintenance services, too, some are nearly maintenance-free, others call for simple maintenance only, and still others demand complicated maintenance services.

The dominant cause of troubles related to the maintenance of all analyzers was, as anticipated, the clogging of flow lines in the sampling system and measuring system ensured from the inherent characteristics of sewage. Especially those lines conveying a small volume of sample were found to be prone to be clogged up readily because of the limited flow velocity and insufficient sectional area. These lines must be made as large in diameter as possible up to the measuring point so as to be able to carry abundant sample, and should also have a structure promising easy flushing operation. The sampling lines at Nagoya plant are commendable in this respect up to the head tanks, but all lines at subsequent treatment stages are not immune from maintenance troubles. If the sampling pipelines are provided with a suitable automatic flushing mechanism, it will certainly contribute to sizable labour-saving.

Continuous water quality monitoring analyzers are often required to be installed in unfavorable conditions such as high humidity or liability to water intrusion. It was found that the tested analyzers were not necessarily installed with due regarded to these unavoidably adverse factors, so that they developed not only electrical troubles but also disjointing of pipes which caused the flow cell to be filled with water. In prevention of such troubles, the layout of internal components of measuring unit rack must be designed with special care. Furthermore, for the purpose of easier and quicker routine inspection, all parts to be checked every day or at short intervals should be gathered on the front of equipment. It was noted that maintenance was made difficult by multitubular piping and by inadequate design, shape or layout of equipment which force the worker to move around it for calibration or inspection, and this gave rise to the occurrence of another troubles. Although improvement of all equipment for completely maintenance-free operation may not be possible, it certainly appears possible to reduce manpower requirement by effecting suitable improvement to various equipment. For easier maintenance and inspection services and quicker detection of troubles, it is advisable to affix a rough sketch showing pipelines, measuring methods, signals and power source on the front surface of each equipment. For all these improvements, cooperation of manufacturers is indispensable and solicited.

To maintain all analyzers in perfect service condition, it is recommended to appoint those with some experience in water quality analysis as exclusive operators of analyzers because nearly all analyzers installed at the two plants call for routine maintenance and inspection services, and the amount of such services is by no means small considering the number of analyzers. The changes in water quality of sewage generally depict the same pattern every day, so that the service condition and malfunction of each analyzer can be cheked with ease from the daily observation record if it is operated by an exclusive operator. The need for such exclusive operated by an exclusive operator. The need for such exclusive operators can also be justified by the fact that some analyzers cannot be operated satisfactorily without the delicate knock which only the exclusive operators can gain through accumulation of skills.

Maintenance cost of analyzers is by no means small even if personnel cost is excluded. NH_3 analyzer, for example, incurs an annual cost of more than 150 thousand yen merely for consumables such as chemicals and tubes. As another conspicuous example of high maintenance cost, it may as well be mentioned that COD analyzer using the acid process is known to require an annual expenditure of more than a million yen only for chemicals.

Utilisation of continuous water quality monitoring equipments in sewage treatment produces immeasurable benefits for overall operation and maintenance of sewage treatment plant in that it makes it possible to know the water quality of raw sewage and effluents at any time. It is to be noted, however, that such benefits can be expected not by simply installing necessary automatic analyzers but by operating each individual analyzer with full understanding of its characteristics and keeping it in perfect fault-free service condition under a well-planned and integrated system of routine inspection and maintenance services.

III. DEVELOPMENT OF AUTOMATED CONTINUOUS P-D, HYARO AND NH_3 ANALYZERS

Pilot or demonstration studies concerning advanced waste treatment are conducted by the Public Works Research Institute of the Ministry of Construction in Kyoto, Nagoya, and Yokosuka.

Of the three plants, studies are conducted on the practicability of Automated Continuous Water Quality Analyzer together with development of advanced water treatment processes in Kyoto and Nagoya.

This report mentions the studies conducted at the plants in Kyoto and Nagoya on colorimetric P analyzers (hereinafter referred to as "P analyzer") and colorimetric NH_3 analyzer (hereinafter referred to as "NH_3 analyzer").

The P analyzers installed in Kyoto and Nagoya are both products of K company. However, the P analyzers and the NH_3 analyzers installed in the Kyoto plant are the products in the developing stage.

Especially, the P analyzers are the first model ever made. On the other hand, as the P analyzers and also the NH_3 analyzers installed in Nagoya in 1976 are improved versions of those in Kyoto, this report will mention the analyzers in Nagoya. However, with regards operating conditions of the analyzers, the report will refer to both Kyoto and Nagoya.

1. P ANALYZER (PHOTO 1)

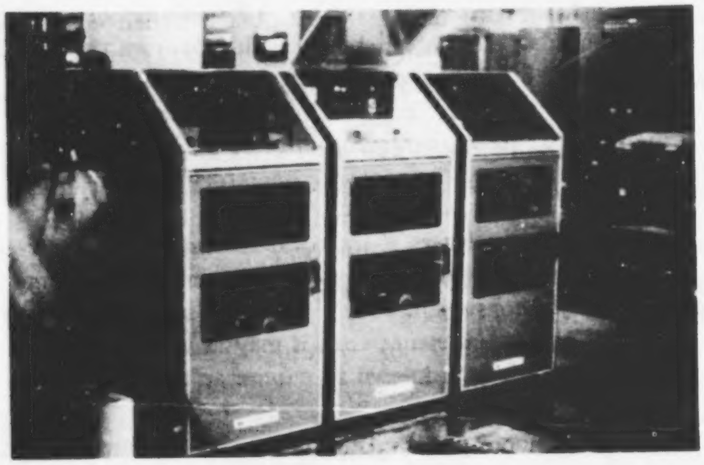
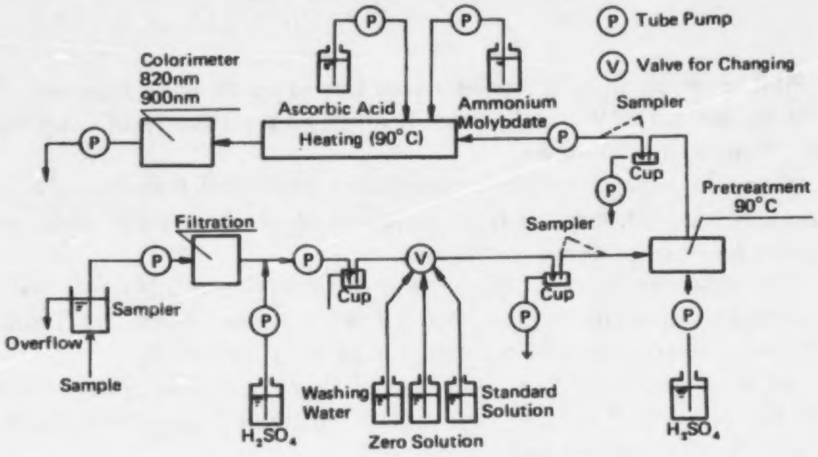


Photo-1 P Analyzer (Nagoya)

The flow sheet of the P analyzer is as shown in Fig. 1. This analyzer is a mechanized version of the manual analyzing method of P-D, hydro. The analyzer can be divided into three stages: Sample filtration, Pre-treatment and Color absorbance measuring stages.

Fig.-1 Flowsheet of P Analyzer



In the Sample filtration stage, the insoluble matters in sample are removed by filter paper. The rolled sheets of filter paper, after filtration, is continuously wound after being dried at about 100 degrees centigrade. And the winding speed is adjustable between 1 and 10 mm per minute depending upon the properties of the insoluble matters. A roll of filter paper is 30 meters long. Therefore, one roll lasts between 2 and 21 days. Then sulfuric acid is injected into the filtrate to prevent contamination of the flow passage.

At the Pretreatment stage, the sample from inside a cup is collected by a vertical movement of a sampler with air which partitions it. Then sulfuric acid is injected into the sample. After mixing polyphosphate and part of organic phosphorous contained in the sample, is subjected to hydrolysis in an incubater maintained at 90 degrees centigrade.

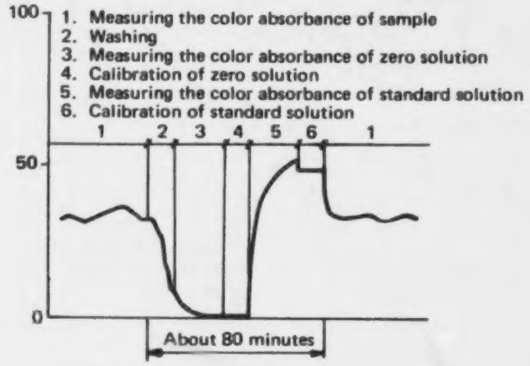
At the Color Absorbance Measuring stage, after releasing air in the pretreated sample from the cup, the sample is again subjected to the vertical motion of the sampler. In a mixing coil, ammonium molibdate is added, and the sample is rolled and mixed. Then in the 90°C incubator, coloring occurs depending upon the orthophosphate contained in the sample. After complete coloring, the sample is colorimetrically analyzed in the flow cell. This unit utilizes the two wave type method to prevent interference caused by color and turbidity in the sample. The P-D, hydro in the sample, measured in such a manner, is sent to either the controller or the recorder by the output of 4 ~ 20 mA or 0 ~ 10 mV.

The response time from the Filteration Stage to the Color Absorbance Measuring Stage is about 30 minutes. The maximum concentration can be set at three ranges of 2.5, 5 and 10 mg·p/l. Washing of the flow passage and calibration are automatically conducted at designated time by a 24-hour timer. Fig. 2 shows the method. After washing, zero solution containing no phosphorous is injected into the piping. And when the indicating value becomes stable, the value is electrically set to "0" This is followed by injection of standard solution containing phosphorous of

50% full scale concentration, and when the indicator shows a stable value, the value is electrically set to 50% of full scale. The time required for washing and calibration is about 110 minutes. About 30 minutes from the start show the sample's measurement values. When a facility is controlled automaticaly based on output from the analyzer, and when washing and calibration is in progress, the output shown immediately before the operation remains unchanged and are transmitted to the control unit. In case of any abnormalty such as dropping of water level in sampling tank, ripping of filter paper, or scaling out, the measurement process is suspended and the alarm is sound. The alarm is sound even when it exceeds the minimim values set in advance.

Daily inspection of flow passage, once weekly regent replenishment, once or twice weekly filter paper changing, and once in between every 2 weeks and one month tube chaning are necessary to maintain them.

Fig.-2 Automated Washing and Calibration



The above mentioned P analyzers in Nagoya are improved versions of those in Kyoto as follows:

1) Filter paper feeding system has become continuous due to simplification of winding mechanism.
2) The filtering direction changed from upflow to downflow to solve the problem of sulfuric acid backflow, injected to prevent contamination of flow passage.
3) The air injection method to divide the sample has been changed form the glass tube utilization method to the sampler vertical motion method. As a result, the intervals between the sample and air became consistent and stable.
4) The colorimeter has been altered from the double beam type to the two wave type analyzing method, and thus the cell has become single instead of double. Then the response time has been shortened.
5) The addition of the Automated Washing Equipment has stabilized measurements. At Kyoto, manual washing is conducted on a twice weekly basis by using a siringe.
6) The flow passage that ran back and forth in the front and rear of the equipment has been grouped together in the front of the unit.

Fig. 3 compares the measurement values of the P analyzer and the manual method (After filtration with No. 5 C filterpape-paper, the P-D, hydro was measured). Sampling was conducted on an once weekly basis during continuous operation of the P analyzers, and measurements were taken for six-months. On the whole, the P analyzer results has shown slightly larger values than the manual method. One of reasons cited can be the sulfuric acid injected to prevent contamination of flow passage after filtration. Fig. 4 shows the results when identical sample was flown into two P analyzers of which only one was injected with sulfuric acid. The Fig. shows that the values of the P analyzer injected with sulfuric acid were higher. This is reasoned to the fact that a part of the injected sulfuric acid has reached the filtration stage and dissolved part of the insoluble phosphorous on the filter.

Fig. 5 shows an example of the automated continuous measurement results of the P analyzer.

Fig.-3 Comparison between P Analyzer and Manual Analysis (Kyoto)

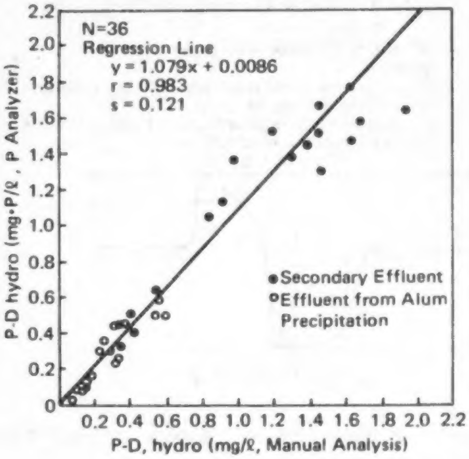


Fig.-4 Effect of H_2SO_4 for Cleaning on Measuring

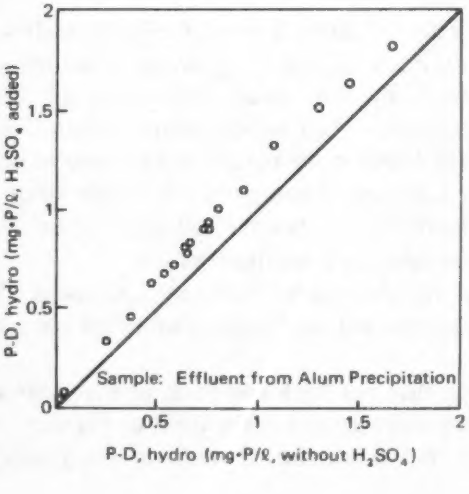
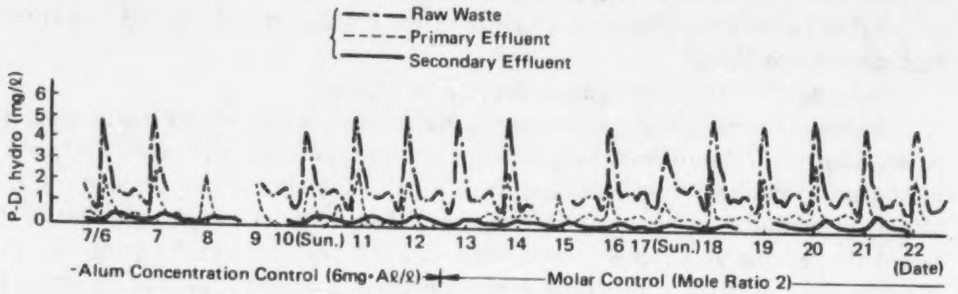


Fig.-5 Variation of P-D, hydro (Alum was dosed into the primary clarifier., Nagoya)



The hitherto developed troubles are as follows:

1) The residual flock containing phosphorous in the alum percipitation effluent was not removed thoroughly by the original filter paper (20μ), and thus the filtrate contained a reasonable amount of insoluble phosphorous. And when supplying paper, the filter paper sometimes was torn. By improving the paper quality and adjusting the pore size to about 1μ, the problems were solved. (Kyoto)

2) Because exchanging and cleaning of tubes at each part were not easy, proficiency of the facility was deteriorated. However, efforts were made by exchanging the flow passage to ease cleaning work. (Kyoto)

3) At first, persulfuric solution was used as a washing agent to clean the pipelines, etc. However, as persulfuric was scattered onto the skin and clothes during manual washing, it was dangerous. Therefore, the washing agent was changed to phosphorous free detergent. The new agent has been bringing about good washing results. (Kyoto)

On the other hand, in Nagoya, the original design required the use of hydrogen peroxide. However, when using only hydrogen peroxide in washing, it was not able to remove scale in the mixing cell immediately before the flow cell. As it was shown that the scale was removable by an alcaline solution, a caustic soda solution was used as a washing agent alternating with hydrogen peroxide solution every week. When using the caustic soda solution, it caused abnormal coloring (especially when phosphorus concentration was high) until being totally discharged and sounded the alarm to suspend operation. (Nagoya)

4) Troubles frequently developed in the filtration stage. And virtually all of the trubles were eliminated by improving the filtration system. Troubles developing in the filtration stage were as follows:

(1) Improper feeding of filter paper. (Kyoto)
(2) Tearing of filter paper. (Kyoto, Nagoya)
(3) Leaking of sample into the filtrate. (Nagoya)
(4) A hole was made in the filter paper due to backflow of sulfuric acid. (Nagoya)

Those were the troubles, but, presently, a total of 5 P analyzers are operating in good conditions both in Kyoto and Nagoya.

A single person can service several P analyzers although it takes about a month for him to be accustomed in handling the units.

Table 1 shows the maintenance costs of the P analyzer and the NH_3 analyzer that are explained later.

Some problems that remain unsolved are as follows:

In case of continuously measuring phosphorous as water quality monitoring, it is necessary to measure not only soluble phosphorous but also insoluble phosphorous. To perform this, development of a unit enabling the measurement of insoluble phosphorous is necessary. Though as the ratio of organic phosphorous contained in the total phosphate in wastewater is small, the hydrolyzable phosphorous is nearly equal to total phosphorus, organic phosphorus can not necessarily ignored in surface water, so development of a unit enabling measurements of organic phosphorous is necessary.

Table-1 Maintenance Costs of P and NH_3 Analyzers (YEN/month·unit)

Item	P Analyzer	NH_3 Analyzer	Remarks
Tube	8,200	8,000	A exchange/two weeks
Filter	1,200	1,200	A exchange/one weeks
Reagent	1,100	2,800	
Total	10,500	12,000	

2. COLORIMETRIC NH_3 ANALYZER (PHOTO 2)

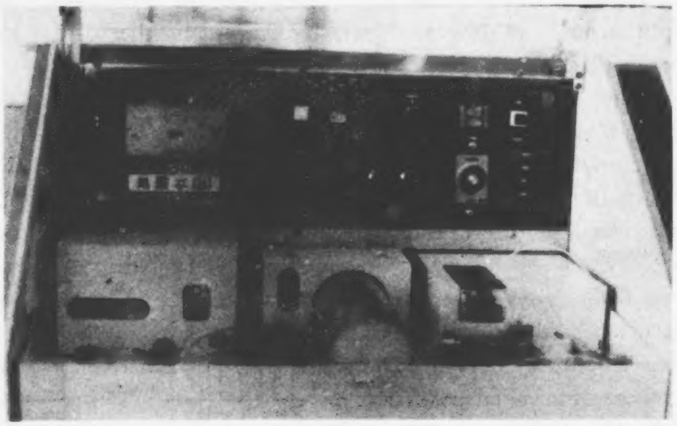


Photo-2 NH_3 Analyzer

The measurement principle of this unit is as follow:

After adding salicylic acid to the sample, when the alkaline isosialic acid is applied, the sample turns green in color. As the degree of the green is proportionate to the ammonia-nitrogen concentration in the sampe, by measuring the absorbance of the green color the amonia-nitrogen conceptration in the sample can be determined. By this method, besides ammonia-nitrogen, mono-chloroamine can also be determined.

The flow sheet of the NH_3 analyzer installed in Nagoya is shown in Fig. 6.

The filteration method, washing and calibration methods, and absorbance measuring method in the NH_3 analyzer are the same as the P analyzer. However, as pretreatment stage is not required in the NH_3 analyzer, the response time is only about 15 minutes, or nearly half of that of the P analyzer.

At maximum measurement concentration of 20 ~ 30 mg·N/l, similarly with the P analyzers, the full scale can be switched into 3 stages and it is possible to operate even at 1/2 or 1/4 of the maximum concentration. When ammonia-nitrogen concentration is more than 20 mg·N/l, a straight calibration line is not obtained. Therefore, when operating in full scale of over 20 mg·N/l, the two fold dilution equipment is employed. The daily routine operation is the same as that of the P analyzer.

Fig. 7 compares actual measurement results of the NH_3 analyzers and the manual method (Indophenol blue method). Fig. 7 shows the measurement results during 18 months of operation, using samples collected at once weekly rate.

Fig. 8 shows an example of continuous measurement results.

Fig.-6 Flowsheet of NH_3 Analyzer

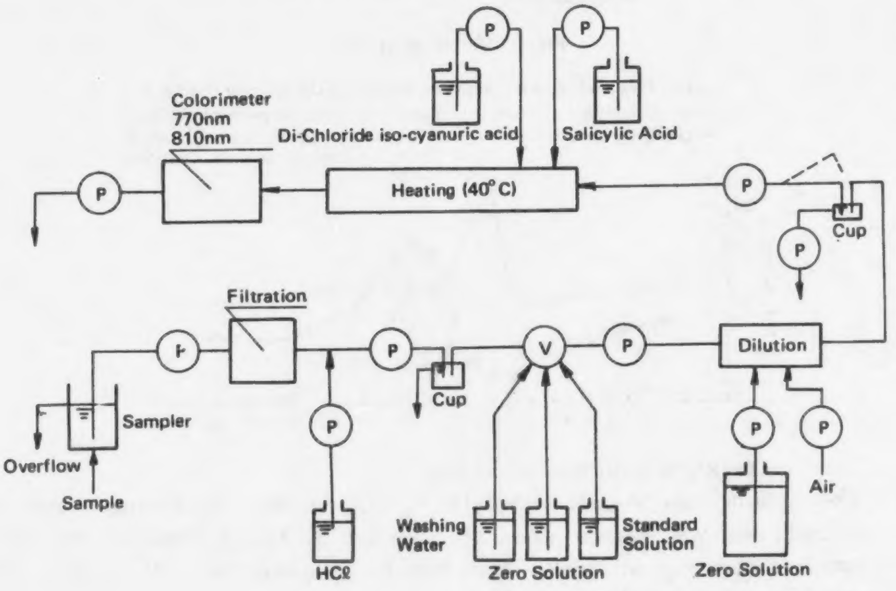


Fig.-7 Comparison between NH_3 Analyzer and Manual Analysis (Kyoto)

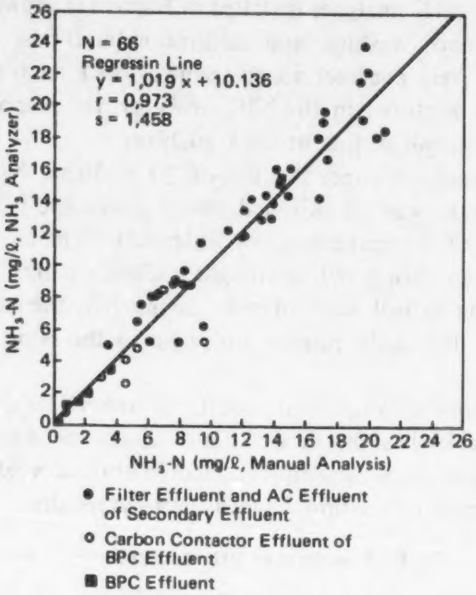
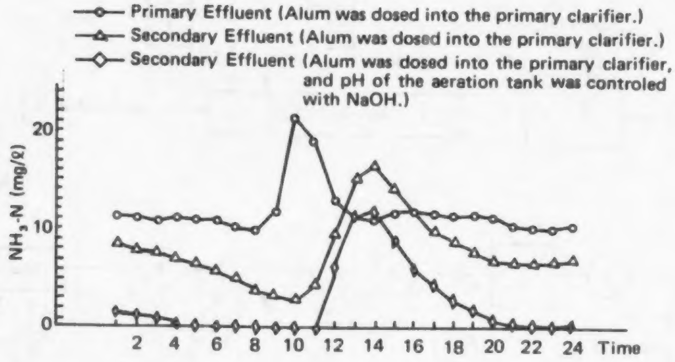


Fig.-8 Variation of NH_3



Trouble developed until now are as follows:

1) The sodium nitroprusside added to expedite rection of coloring regent of salicylic acid was instable when pH exceeded 8. This deteriorated the measurement accuracy gradually. However, by adjusting the pH correctly, the problem was solved. (Kyoto)

2) As the open type standard solution preservation container was used, the water in the standard solution evaporated. There were times when the 10 mg·N/l standard solution condensed to 13 mg·N/l 10 days later, sealedtype containers are now used. (Kyoto)

3) Also there were problems similar to those in the P analyzers.

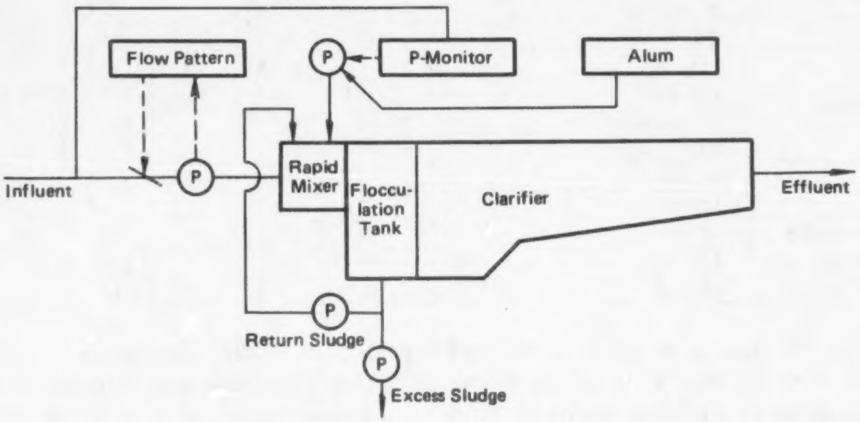
Presently, a total of 6 NH_3 analyzers are functioning normally at the plants in Kyoto and Nagoya. Table 1 shows the daily maintenance costs.

IV. ALUM DOSING METHOD IN ALUM PRECIPITATION

At the Kyoto pilot plant, experiments were conducted to compare the alum constant dosing method with the molar control method, that is, alum is dosed into the influent in proportion to the dissolved phosphorous concentration (P-D). The flow diagram of the precipitation facility is as shown in Fig. 1. And the experimental conditions and results are as shown in Table 1. The influent was effluent form conventional activated sludge. The influent flow was 300 m^3/day (in case of fluctuations in flow, the daily average influent). The overflow rate of the clairifer was 30 m^3/day.

Fig.-1 Flow Diagram of Alum Precipitation

Rapid Mixer	1,000W x 1,000L x 800D
Flocculation Tank	1,000W x 2,000L x 3,600D
Clarifier	1,000W x 10,000L x 3,600D



EXPERIMENTAL CONDITIONS – See Table 1 –

Experiment numbers 1 ~ 3 were constant doxing method, and experiment numbers 4 ~ 6 were molar control method.

The amount of alum doze in the constant dosing method was set at 5.9 mg·Al/liter. In case of molar control method, based on the phosphorous monitor output that continuously measured P-D of the influent, the alum doxing pump was regulated to maintain the molar ratio at 6 of Al against P-D of the influent.

The experiments on the constant dosing method and the molar control method were diversified as follows:

Experiments 1 and 4: Constant influent flow without return sludge.
Experiments 2 and 5: Constant influent flow with return sludge.
Experiments 3 and 6: Varying influent flow with reutrn sludge.

Table -1 Experimental Conditions and Results of Alum Precipitation at Kyoto P.P.

Item	Number of Experiment	1	2	3	4	5	6
Conditions	Flow	Constant	Constant	Varying	Constant	Constant	Varying
	Alum Dosing Method	Constant	Constant	Constant	Molar Control	Molar Control	Molar Control
	Sludge Return Rate (%)	0	4.2	5.1	0	4.0	4.9
	Excess Sludge Rate (%)	0.6	0.7	0.7	0.7	0.7	0.7
	Alum Dosing (mg·Al/l)	5.9	5.9	5.9	7.1	7.3	6.8
	Mole Ratio (Al/P-D)	4.8	4.6	5.7	6	6	6
P (mg·P/l)	Influent	1.69	1.83	1.44	1.50	1.51	1.44
	Effluent	1.01	1.08	0.80	0.80	1.08	0.79
	Excess Sludge	26.7	74.7	35.8		23.9	37.2
	Mixed Liquor		9.40	6.27		3.20	4.95
	Removal (%)	40.2	41.0	44.5	46.7	28.4	45.1
P-D (mg·P/l)	Influent	1.40	1.48	1.18	1.36	1.39	1.30
	Effluent	0.44	0.32	0.23	0.30	0.25	0.26
	Excess Sludge	0.19	0.39	0.22		0.29	0.27
	Mixed Liquor		0.87	0.47		0.66	0.32
	Removal (%)	68.6	78.4	80.5	77.9	82.0	80.0
SS (mg/l)	Influent	9.7	7.7	12.6	6.1	5.8	8.9
	Effluent	24.8	17.6	16.2	13.3	19.7	15.2
	Excess Sludge	1690	2190	1740		640	1450
	Mixed Liquor		188	155		74	112
pH	Influent	7.2	7.1	7.0	7.1	7.1	7.1
	Effluent	6.9	7.0	6.9	6.9	6.9	6.9
M-Alkalinity (mg·$CaCo_3$/l)	Influent	159	145	150	144	143	144
	Effluent	135	129	117	123	113	118
Temperature	Effluent	20.5	16.1	19.1	15.1	13.4	16.7

By performing the above mentioned experiments, studies were made on alum dosing methods, effect of return sludge, influence of variation of influent flow. The experiment results as shown in Table 1 are average values for each experiment. Also, the analysis of influent and effluent was conducted by using the composite samples of each day. Other samples were collected at 10 in the morning everyday.

EXPERIMENTAL RESULTS AND DISCUSSION

According to Table 1, the removal rate of phosphorous (P) was low ranging between 40 and 45%, except for Experiment 5. In contrast, the P-D removal rate was about 80%, and in all cases SS in the effluent was higher than SS in the influent. From these facts, the lowliness of the phosphorous removal rate can be considered to be caused by the lowliness of the insoluble phosphorous removal rate. Therefore, it is necessary to conduct studies on the improvement of the clarifier.

Results of P-D removal are shown in Fig. 2 concerning Experiments 1 ~ 3, and in Fig. 3 concerning Experiments 4 ~ 6.

Fig.-2

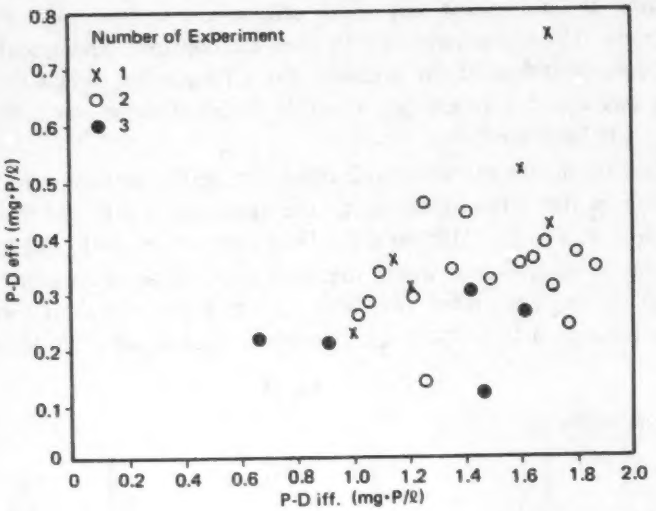
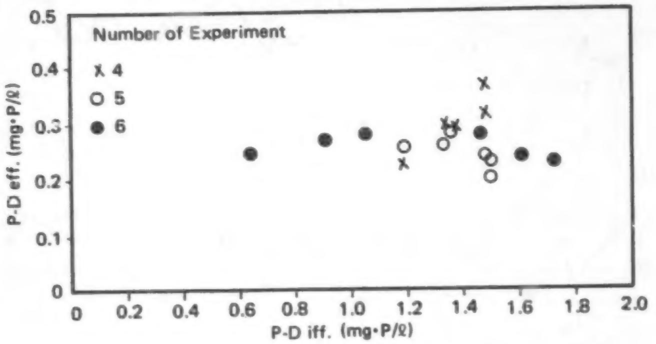


Fig.-3



From Table 1, Fig. 2 and Fig. 3, the followings can be considered concerning the P-D removal:

1) In both the constant dosing method and the molar control method, it was possible to lower P-D concentration in the effluent by the return sludge.

   Also, it lessened the fluctuations of P-D concentration in the effluent. This is because the alum sludge still had the ability to remove phosphorous.

   In Table 1, Mixed Liquor is a combination of influent and return sludge before alum dosing. In other words, Table 1 shows that between 40 and 75 percent of P-D in the influent were removed by the return sludge. Accordingly, the return sludge lowered the P-D concentration in the influent, and as variations became smaller, it lowered P-D concentration in the effluent to further minimize fluctuation.

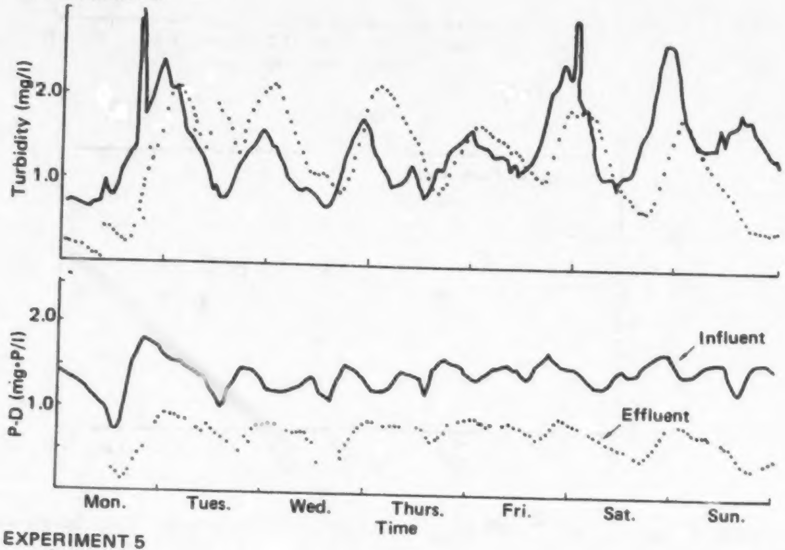
2) The molar control method prevented variation of P-D concentration in the effluent by either daily average (Fig. 2, 3) or continuous measurement (Fig. 4, 5) than the constant dosing method even when sludge was returned. However,

according to Fig. 2, 3, the daily average value of P-D concentration in the effluent did not show any clear differences between the two alum dosing methods. The molar ratio of P-D/Al in the constant dosing method was 6 when P-D concentration of the influent was 1.13 mg·p/l. When the P-D concentration was about 1.13 mg·p/l, the P-D in the effluent was a little less than 0.3 mg·p/l in both methods.

3) Both alum dosing methods with return sludge did not cause increase of the P-D loading in the effluent owing to the variation in influent flow. (See Table 1 and Fig. 4, Fig. 5) Although this was considered quite natural in the molar control method, it was worth mentioning in the alum constant dosing method. So, it can be concluded that with regards P-D removal in alum precipitation, there is no need to conduct molar control method when sludge is returned.

Fig. -4

EXPERIMENT 2



EXPERIMENT 5

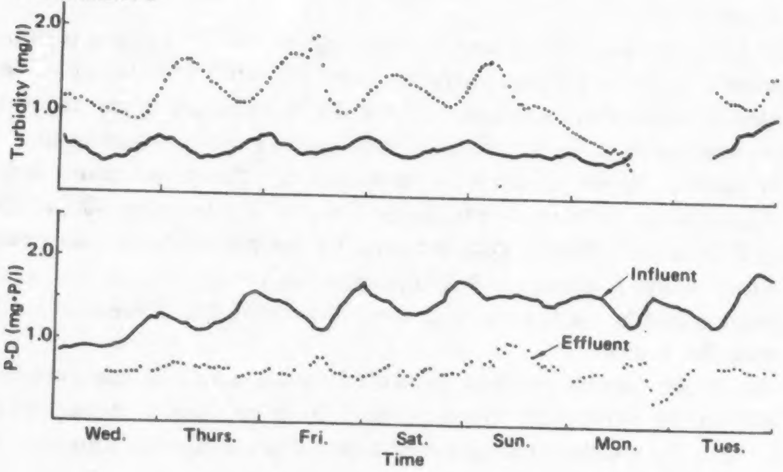
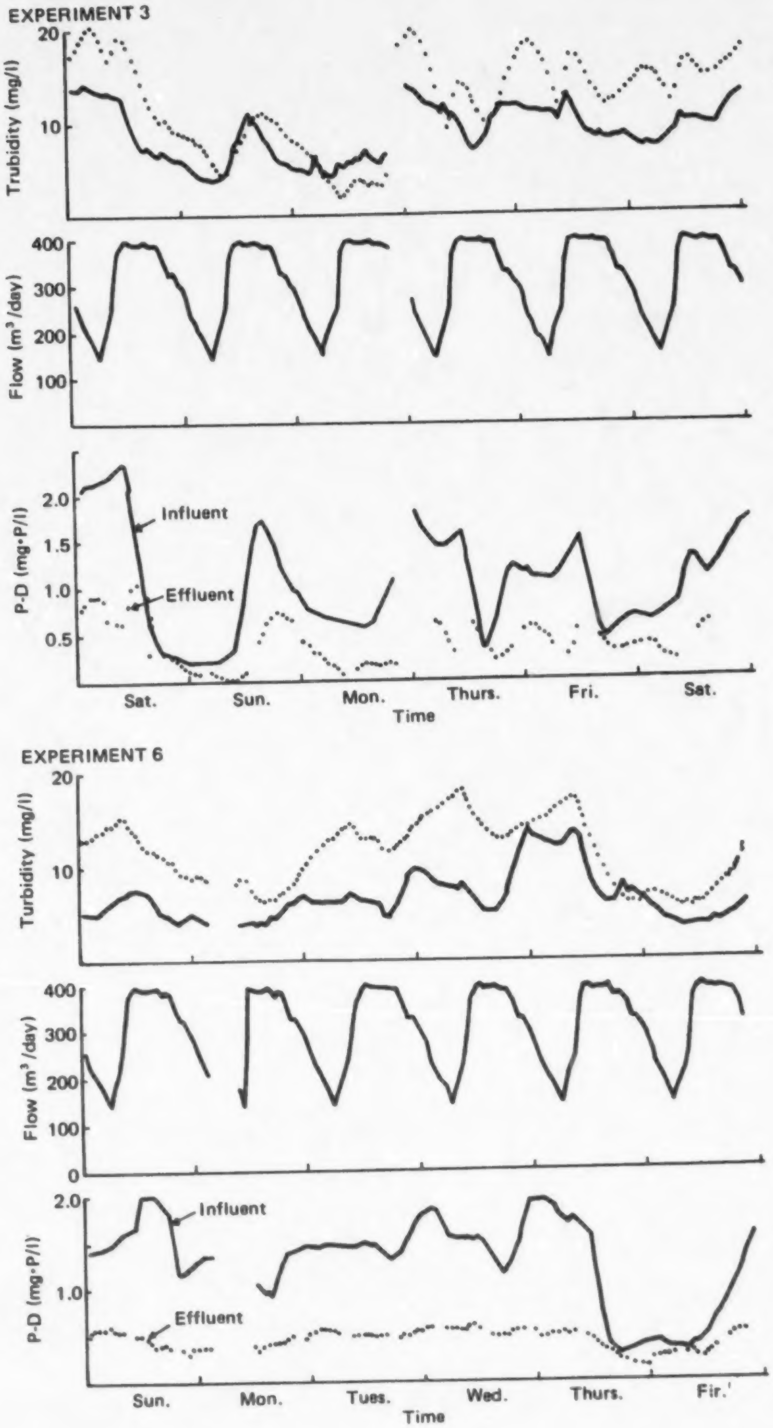


Fig. -5


EXPERIMENT 3
Trubidity (mg/l)
20
10
Flow (m³/day)
400
300
200
100
P-D (mg-P/l)
2.0
1.5
1.0
0.5
Influent
Effluent
Sat.
Sun.
Mon.
Thurs.
Fri.
Sat.
Time
EXPERIMENT 6
Turbidity (mg/l)
20
10
Flow (m³/day)
400
300
200
100
0
P-D (mg-P/l)
2.0
1.0
Influent
Effluent
Sun.
Mon.
Tues.
Wed.
Thurs.
Fir.'
Time

PAPERS FOR THE SIXTH US/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

Washington D.C.
November 1 – 2, 1978

Ministry of Construction
Government of Japan

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CHAPTER I

RECENT TOPICS OF WATER POLLUTION CONTROL IN JAPAN

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I CURRENT STATE OF THE WATER POLLUTION

In general, the state of water pollution has tended to improve in recent years, reflecting the vigorous enforcement of effluent control measures which have been continuously strengthened.

The ratio of samples which met the environmental quality standards relating to human health (cadmium,cyanide and seven other toxic substances) continued to rise in 1976, as in 1975, reaching the very high value of 99.91 percent; in neither year was any alkyl mercury or organic phosphorus detected in samples, in addition to this, hexavallent chromium was not detected in samples in 1976.

The recent trends in BOD(or COD for lakes and coastal water bodies) which are major items of the standards relating to the living environment tend to show an improvement in water pollution on the whole.

Next, with respect to the attainment of the environmental water quality standards for BOD or COD in 2,586 water areas (2,024 rivers, 82 lakes and 480 coastal water bodies) categolised by virtue of a Cabinet decision taken in September 1970 and also by virtue of a Cabinet Order issued in May 1971, 60.5 percent (1,164 rivers, 34 lakes and 366 coastal water bodies; total, 1,564 water areas) had attained the environmental standards as of the end of 1976, showing an improvement over the 59.6 percent attainment of 1975. Specifically, the ratios of attainment in 1976 stood at 57.5 percent of rivers (57.1 percent in 1975), 41.5 percent of lakes (38.6 percent in 1975) and 76.3 percent of coastal water bodies (72.4 percent in 1975). Also, the ratio of attainment of the environmental standards by major rivers which flow through large cities and coastal water areas which are adjacent to large cities remained lower than of other water areas, leaving room for still further improvement.

The Seto Inland Sea is a scenic area of rare beauty and an invaluable storehouse of marine resources, and as such pollution control measures to check the deterioration of its water quality have become a problem of the utmost importance. The enactment of the Seto Inland Sea Conservation Law in 1973 and the implementation of various measures have initiated a tendency toward improvement, but the still-continuing occurence of "red tides" and contamination by spilled oil caused by accident (for example collision of crude oil carriers) are evidece of the need for appropriate countermeasures to meet the different states of pollution.

2 STATUS QUO OF WATER POLLUTION CONTROL LAW AND SETO INLAND SEA CONSERVATION LAW IN THEIR ENFORCEMENT

Under the reguration of Water Pollution Control Law, the person who discharges effluent from factories or establishment into the public water area should submit a report, whenever he intends to install any Specified Facilities, to the prefectural governor (including Works-Entrusted Mayor of Cities which are designated in the Cabinet Order).

Much the same, under the regulation of Seto Inland Sea Conservation Law, the person who discharges effluent from factories or establishment into the public water area, wihtin the area of Osaka Prefecture and other ten prefectures relating to Seto Inland Sea (excluding the area where has no relationship to the water pollution of Seto Inland Sea such as the geographical portion which faces to Sea of Japan) shall obtain permission of or should submit a report to the governor concerned (including works-entrusted mayor of cities which are designated in the Cabinet Order), whenever he intends to install any Specified Facilities.

2.1 SPECIFIED FACTORIES

As of March 31,1977, 250,801 Specified Factories had been submitted reports to the governors of the competent prefectures under the provision of Water Pollution Control Law (including 4,062 Specified Factories to which the permits had been issued by the competent governors of prefectures under the provision of Seto Inland Sea Conservation Law).

The Effluent Standards relating to preservation of living environment (such as pH) issued by the Ordinance of the Prime Minister's Office are applied to the Specified Factories which discharge 50 cubic metres or more effluent per day on an average, and as of March 31, 1977, 28,254 reports from those factories had been received by the governors of prefectures.

On the contrary, the Effluent Standards relating to harmful substances (such as cadmium) are applied to all the Specified Factories whatever the quantities of their effluent are. Among the 222,547 Specified Factories which discharge less than 50 cubic metres per day on an average, those which discharge harmful substances are 7,338 and this is 3 percent of the total. So at least, the total number of Specified Factories which are applied with so called Uniform Effluent Standards is 35,592 and this is 14 percent of the total. Moreover, among the Specified Factories which discharge 50 cubic metres or more of effluent per day on an average, those which discharge harmful substances are 3,592 and this is 1 percent of the total.

Table 1 shows ten major types of industries which hold higher ranks in number as the Specified Factories as of March 31, 1977.

Also, Table 2 shows " Specified Facilities" under the provision of the Cabinet Order of Water Pollution Control Law, for reference.

As to the sewage treatment plants which submitted the reports to the governors of prefectures according to the regulation of the Water Pollution Control Law (including nine plants to which permissions issued by the governors of prefectures concerned under the provisions of Seto Inland Sea Conservation Law) are 467 in number as of March, 1977.

Several Specified Facilities have been added during the period from October 1972 to June 1976. Those facilities are as follows.

Livestock breeding (October 1, 1972, No. 1-2)
Desizing facilities of textile industry (December 1, 1974,

(i) of No. 19)

Lodging service (December 1, 1974, No. 66-2)

Research,determination,measurement or professional education for science and technology (December 1, 1974, No. 71-2)

Domestic and industrial water supply (June 1, 1976, No.64-2)

Central wholesale market of aquatic products (June 1,1976, No. 69-2)

Also, the Environment Agency intends to add new specified facilities at need for coming years. For instance, hospitals and waste treatment facilities will be added in near future.

2.2 MORE STRINGENT EFFLUENT STANDARDS

Under the provision of the Water Pollution Control Law, when there is any Public Water Area under a prefectural jurisdixion for which the effluent standards as provided for is recognised to be insufficient for protecting human health or for preserving the living environment, the prefecture may establish more stringent standards than the standards established by the Government (Water Pollution Control Law, Article 3).

All of the prefectures (47) have established more stringent effluent standards as of March 31, 1977.

As to the contents of "more stringent effluent standards", prefectures which have the standards relating to harmful substances are 23 prefectures, while the remainder 24 prefectures have established more stringent standards only for the items relating to the preservation of living environment, as of March 31, 1977.

Also, as to the factories which discharge less than 50 cubic metres per day on an average, the effluent standards established by the Government are not applied in principle, but 32 prefectures deal with even those factories as the objects to be controled by the more stringent standards.

2.3 STATE OF ENFORCEMENT OF SETO INLAND SEA CONSERVATION LAW

Seto Inland Sea Conservation Law was unanimously enacted by the Diet in 1973 in view of the special importance of this area as a treasure house of scenic beauty and fishery resources. In line with the Law, a series of special measures have been taken to protect the environment of Seto Inland Sea.

These measures include, among others, (a) to make a basic plan for the conservation of the environment of Seto Inland Sea, (b) to reduce, within the period of three years, the Chemical Oxygen Demand load upon the Seto Inland Sea from the industrial sector to one half of the level of 1972, (c) to require special permission for the new setting up of certain types of industrial facilities, and (d) to ensure special consideration on the peculiar importance of the Seto Inland Sea.

Initially, this law was planned to lapse after three years in accordance with the Article 4 of supplementary provisions of this law. Afterwards, in 1976, this limitation of period was prolonged two more years -- until November 1, 1978.

As to the goal for the reduction of COD loading, we have got a big success beyond the goal. Namely, the goal was to reduce the COD load upon the Inland Sea from 1,345 tons in 1972 to its one half, i.e., 673 tons, while the result was 459.5 tons in November 1976. It means 131.7 percent attainment against the aim.

As to the special consideration on land reclamation in Seto Inland Sea area, before the enforcement of this law, for instance in the period from January 1, 1970 to November 1, 1973, the average anual reclamation area was 2,190 hectare, while after the enforcement of this law, in the period from November 2, 1973 to October 31, 1977, the average anual reclamation area was 540 hectare, and this means three quarter reduction of pace.

Also, as to the basic plan for for the conservation of the environment of Seto Inland Sea, it was made by the Environment Agency through the discussion of Seto Inland Sea Environmental Conservation Council and decided by the Cabinet Meeting, in April 1978.

3 AMENDMENT OF SETO INLAND SEA CONCERVATION LAW AND WATER POLLUTION CONTROL LAW

As stated before, the Seto Inland Sea Conservation Law was expected to expire in November 1978, strong views have been expressed from various sectors calling for necessary legislative action to extend the validity of the Law.

Inthe meanwhile, the water quality in the public water areas in Japan has been generally improved as stated before. But it is still difficult to attain the environmental water quality standards in some large scale closed/semi-closed water bodies the Seto Inland Sea, Tokyo Bay and Ise Bay.

Under such circumstances, a new Amendment Bill was proposed to extend the validity of the Seto Inland Sea Conservation Law and to establish a Total Mass Effluent Control system for water pollution control in the large size closed/semi-closed water bodies, under which total amount of pollutant load is to be regulated, as well as to conserve the environment of the natural beaches.

Thus, on June 7 the National Diet passed the bill for revision of the Special Law for Conservation Of the Environment of the Seto Inland Sea and the Water Pollution Control Law.

3.1 AMENDMENT OF SETO INLAND SEA CONSERVATION LAW

The extention of validity of the Law is now realised by the enactment of this bill, by which the supplemental provisions limiting the implementation period for the Special Law for Conservation of the Environment of the Seto Inland Sea are abolished.

The highlight of the amendment is as follows:

Firstly, establishment a new provision for Prefectural Plan was introduced to realise the effectiveness of various countermeasures taken in the competent prefectures according to the geographical and social characterristics.

Secondlly, Total Mass Effluent Control system was introduced for the region of Seto Inland Sea. This is a measure to take over the measures of reduction of COD load from the industrial sector (extended for COD load from the domestic sector in the amendment).

Thirdly, new measures directed to the prevention of damage due to eutrophication, like the "red tides" in the Seto Inland Sea, are incorporated in the Water Pollution Control Law. The amendment makes it possible to regulate the inflow of phosphorus and other substances which are to be specified by a Cabinet Order as eutrophication substances. More specificaly, the prefectural governors in areas concerned are now empowered to give administrative guidance to factories when it is deemed necessary to reduce the volume of such eutrophication substances in their effluent. This guidance is to be given in accordance with guidlines to be set by the Director-General of the Environment Agency.

Fourthly, new measures related to the preservation of natural beaches used for sea bathing and other forms of recreation are included. The prefectural government bodies concerned are now authorised to designate such natural beaches as requiring preservation, to require that the building of structures and other activities in the area be reported to the prefectural authorities in advance and to give necessary guidance regarding such activities.

Fifthly, a new provision introduced to prevent oil pollution in the Seto Inland Sea caused by marine accidents; regulations are requesting the Government to make efforts to take necessary measures for strengthening of guide and control, preparation of preventing and removal system, concerning large quantities of oil discharge caused by marine accident and removal of spilled oil.

3.2 AMENDMENT OF WATER POLLUTION CONTROL LAW

As for the revision of the Water Pollution Control Law to set up a system for regulating the total amount of pollutants in effluent, the Prime Minister is now expected to work out a basic policy for the reduction of the total pollution load for pollutants which are to be determined by a Cabinet Order (limited to COD for the time being), with respect to river basins (Specified Areas) of arge size closed/semi-closed water bodies where attainment and maintenance of the environmental water quality standards is difficult (Seto Inland Sea , Tokyo Bay, etc.).

Next, the prefectural governor concerned is to work out a programme in line with the said basic policy for the reduction of the total effluents, and according to this programme, to work out mass regulation standards to be observed by factories and other workshops larger than a certain scale which are located in the Specified Areas. Unlike the effluent standards enforced heretofore which regulate only the concentration of pollutants, the mass regulation standards will set the permissible level of the pollutant load emitted by factories and other workshops concerned.

Moreover, if necessary for carrying out the programme for the reduction of the effluents, the prefectural governor is authorised to give guidance even to persons other than the factories and other workshops which are required to observe the mass regulation standard.

In order to ensure the effectiveness of the total mass regulation system, the amendment also includes provisions for the issuance of orders to enforce programme change or improvement, and establishes the obligation to measure and record the pollution load.

The amendment is to take effect at a date within a year from the day of its promulgation (June 13).

Also, the goal date of the attainment of the aimed level of water quality in the competent water bodies by this total mass control system are scheduled five years after the date that this amendment is to take effect.

Table 1 Major types of industries hold higher ranking in number as Specified Factories

Ranking	Types of industries *	Number **	Ratio of exceeding 50 cubic metre per day of effluent discharge
1	Lodging service (No. 66-2)	72,908 (29.1%)	5.3 %
2	Livestock breeding (No. 1-2)	36,746 (14.7%)	1.5 %
3	Laundry (No. 67)	26,474 (10.6%)	1.0 %
4	Bean foods manufacturing (No. 17)	21,889 (8.7%)	0.7 %
5	Automatically washing facilities of car (No. 71)	13,463 (5.4%)	1.8 %
6	Sea foods manufacturing (No. 3)	10,518 (4.2%)	9.2 %
7	Night soil treatment plant (No. 72)	7,703 (3.1%)	87.5 %
8	Textile industry (No. 19)	5,091 (2.0%)	31.8 %
9	Acid and alkali treatment facilities of metal surface (No. 65)	5,049 (2.0%)	33.1 %
10	Soft drink manufacturing and brewery (No. 10)	4,683 (1.9%)	11.3 %

Note ; * ( ) shows the number listed in "Table 1" annexed to the Cabinet Order of Water Pollution Control Law.

** ( ) shows the percentage of the competent Specified Factories in number against the total number of Specified Factories, as of March 31, 1977.

Table 2 The list of Specified Facilities

(1) MINING AND COAL WASHING
- (a) ore separation facilities,
- (b) coal dressing facilities,
- (c) neutrlisation and sedimentation facilities of mine water,
- (d) solids separation facilities from water used for digging.

(1)-2 LIVESTOCK BREEDING
- (a) pig shed facilities (excluding the facilities installed in the shed with the total area of less than 50 square metre),
- (b) cattle shed facilities (excluding the facilities installed in the shed with the total area of less than 200 square metre),
- (c) horse shed facilities (excluding the facilities installed in the shed with the total area of less than 500 square metre).

(2) MEAT PACKING AND POULTRY PROCESSING
- (a) initial preparation facilities,
- (b) washing facilities,
- (c) cooking facilities.

(3) SEA FOODS MANUFACTURING
- (a) initial preparation facilities,
- (b) washing facilities,
- (c) dehydration facilities,
- (d) screening facilities,
- (e) cooking facilities.

(4) TINNED AND FROZEN VEGETABLES AND FRUITS MANUFACTURING
- (a) initial preparation facilities,
- (b) cleaning facilities,
- (c) pressing facilities,
- (d) cooking facilities.

(5) MISO, SOY-SOURCE, EDIBLE AMINO ACID, GLUTAMIC ACID, VEGETABLE SOURCES AND VINEGAR MANUFACTURING
- (a) initial preparation facilities,
- (b) cleaning facilities,
- (c) boiling facilities,
- (d) concentration facilities,
- (e) finishing facilities,
- (f) straining facilities.

(6) WHEAT FLOUR MANUFACTURING
- washing facilities

(7) SUGAR MANUFACTURING
- (a) initial preparation facilities,
- (b) washing facilities,
- (c) filtration facilities,
- (d) separation facilities,
- (e) refining facilities.

(8) BAKERY AND CONFEXIONARY
- bean-jam processing facilities

- (9) RICE CAKE AND MALT MANUFACTURING
  - washing facilities
- (10) SOFT DRINK MANUFACTURING AND BREWERY
  - (a) initial preparation facilities,
  - (b) cleaning facilities,
  - (c) extraxion facilities,
  - (d) straining facilities,
  - (e) boiling facilities,
  - (f) stilling facilities.
- (11) FEED STAFF AND ORGANIC FERTILISER MANUFACTURING
  - (a) initial preparation facilities,
  - (b) washing facilities,
  - (c) pressing facilities,
  - (d) vacuum concentration facilities,
  - (e) water washing deodorisation facilities.
- (12) OIL AND FAT MANUFACTURING
  - (a) initial preparation facilities,
  - (b) washing facilities,
  - (c) pressing facilities,
  - (d) separation facilities.
- (13) YEAST MANUFACTURING
  - (a) initial preparation facilities,
  - (b) washing facilities,
  - (c) separation facilities.
- (14) STARCH MANUFACTURING
  - (a) soaking facilities,
  - (b) washing facilities,
  - (c) separation facilities,
  - (d) waste pits.
- (15) DEXTROSE MANUFACTURING
  - (a) initial preparation facilities,
  - (b) filtration facilities,
  - (c) refining facilities.
- (16) NOODLE MANUFACTURING
  - boiling facilities
- (17) BEAN FOODS MANUFACTURING
  - boiling facilities
- (18) INSTANT COFFEE MANUFACTURING
  - extraxion facilities
- (19) TEXTILE INDUSTRY
  - (a) scouring facilities,
  - (b) by-product processing facilities,
  - (c) soaking facilities,
  - (d) finishing facilities,
  - (e) silket machine,
  - (f) bleaching facilities,
  - (g) deying facilities,
  - (h) chemical treatment facilities
  - (i) desizing facilities
- (20) WOOL SCOURING AND WASHING
  - (a) wool scouring and washing facilities,
  - (b) carbonising facilities.
- (21) SYNTHETIC TEXTILE MANUFACTURING

(a) spinning facilities,
(b) chemical treatment facilities,
(c) recovery facilities.

(22) CHEMICAL FINISHING OF WOODS
(a) wet barker,
(b) chemical soaking facilities.

(23) PULP AND PAPER MANUFACTURING
(a) soaking facilities,
(b) wet barker,
(c) chipper,
(d) digester,
(e) accumulator for digester waste,
(f) chip refiner and pulp refiner
(g) bleaching facilities,
(h) paper mill,
(i) cellophane paper mill,
(j) wet fibre plate facilities,
(k) waste gas washing facilities.

(24) FERTILISER MANUFACTURING
(a) filtration facilities,
(b) separation facilities,
(c) water jet breaking facilities,
(d) waste gas washing facilities,
(e) wet dust collector.

(25) SODIUM HYDROXIDE AND POTASSIUM HYDROXIDE MANUFACTURING (MERCURY ELECTROLYSIS)
(a) electrolyte refining facilities,
(b) electrolising facilities.

(26) INORGANIC PIGMENT MANUFACTURING
(a) washing facilities,
(b) filtration facilities,
(c) centrifuger (cadmium and its compounds)
(d) water flushing separator (ultra-marine)
(e) waste gas washing facilities.

(27) INORGANIC CHEMICALS MANUFACTURING EXCLUDING ITEMS OF 25 AND 26
(a) filtration facilities,
(b) centrifuger,
(c) sulfur dioxide gas cooling and washing facilities (sulfuric acid),
(d) washing facilities (activated carbon and carbonated disulfur),
(e) hydrochloric acid regenerating facilities (silicate anhydrous),
(f) reactor (cyanides),
(g) absorber and sedimentation facilities (iodines),
(h) sedimentation facilities (saline magnesia),
(i) water flushing facilities (bariumates),
(j) waste gas washing facilities,
(k) wet dust collector.

(28) ACETYLENE DERIVATIVES MANUFACTURING (CARBIDE PROCESS)
(a) wet acetylene generation facilities,

- (b) washing facilities and still (acetate ester),
- (c) methyl alcohol still (polyvinyl alcohol),
- (d) still (acrylic acid ester),
- (e) vinyl chloric monomer washing facilities,
- (f) chlorprene monomer washing facilities.

(29) COAL TAR PRODUCTS MANUFACTURING
- (a) sulfuric acid washing facilities of benzene relates,
- (b) waste pits,
- (c) tar sodium sulfonate reactor.

(30) FERMENTATION INDUSTRY EXCLUDING ITEMS OF 5, 10 AND 13
- (a) initial preparation facilities,
- (b) still,
- (c) centrifugal decanter,
- (d) filtration facilities.

(31) METHANE DERIVATIVES MANUFACTURING
- (a) still (methyl alcohol and 4-chloromethane),
- (b) refining facilities (formaldehyde),
- (c) washing and filtration facilities,

(32) ORGANIC PIGMENT MANUFACTURING OR SYNTHETIC DYES MANUFACTURING
- (a) filtration facilities,
- (b) water washing facilities,
- (c) centrifugal decanter,
- (d) waste gas washing facilities.

(33) SYNTHETIC PLASTIC MANUFACTURING
- (a) condensation reactor,
- (b) water washing facilities,
- (c) centrifugal decanter,
- (d) settling facilities,
- (e) cooling gas washer and still (fluorides plastics),
- (f) diluent still (polypropylene),
- (g) diluent still (polyethylene),
- (h) acid and alkali treatment facilities (polybutane),
- (i) waste gas washing facilities,
- (j) wet dust collector.

(34) SYNTHETIC RUBBER MANUFACTURING
- (a) filtration facilities,
- (b) dehydration facilities,
- (c) washing facilities,
- (d) latex concentration facilities,
- (e) sedimentation facilities (styrene-butadiene,nitrile-butadiene and polybutadiene-gum).

(35) ORGANIC GUM CHEMICALS MANUFACTURING
- (a) distilation facilities,
- (b) waste gas washing facilities,
- (c) wet dust collector.

(36) SYNTHETIC DETERGENT MANUFACTURING
- (a) acid washing and separating facilities,
- (b) waste gas washing facilities,
- (c) wet dust collector.

(37) PETROCHEMICAL INDUSTRIES (CARBOHYDRATE AND ITS DERIVATIVES) EXCLUDING ITEMS FROM 31 TO 36 AND 51

- (a) washing facilities,
- (b) separation facilities,
- (c) filtration facilities,
- (d) distilation and rapid cooling facilities (acrylonitrile),
- (e) distilation facilities (acetoaldehyde, acetone, terephthalic acid and tolylene diamine),
- (f) acid and alkali tratment facilities (alkylbenzene),
- (g) distilation facilities and sulphuric acid concentration facilities (isopropyl alcohol),
- (h) distilation and condensation reactor (ethylene oxide or ethylene glycol),
- (i) condensation reactor and distilation facilities (2-ethyl hexyl alcohol or isobutyl alcohol),
- (j) acid and alkali treatment facilities (cyclohexane),
- (k) gas cooling and washing facilities (phthalic anhydride),
- (l) acid or alkali treatment facilities and methyl-alcohol distilation facilities (normal paraffin),
- (m) saponification facilities (propylene oxide or propylene glycol),
- (n) steam condenser (methylethyl ketone)
- (o) reactor and methyl alcohol recovery facilities (methyl-meta-acrylate monomer),
- (p) waste gas washing facilities.

(38) SOAP MANUFACTURING
- (a) initial preparation facilities,
- (b) salting out facilities.

(39) HYDROGENATED OIL MANUFACTURING
- (a) alkali conditioning facilities,
- (b) deodorisation facilities.

(40) FATTY ACIDS MANUFACTURING
- distilation facilities

(41) PERFUMERY MANUFACTURING
- (a) washing facilities,
- (b) extraxion facilities.

(42) GELATINE AND GLUE MANUFACTURING
- (a) initial preparation facilities,
- (b) lime soaking facilities,
- (c) washing facilities.

(43) PHOTO SENSITIVE GOODS MANUFACTURING
- washing facilities

(44) NATURAL RESIN MANUFACTURING
- (a) initial preparation facilities,
- (b) dehydration facilities.

(45) WOODS CHEMICAL MANUFACTURING
- furfural distilation facilities

(46) ORGANIC CHEMICALS MANUFACTURING EXCLUDING ITEMS FROM 28 TO 45
- (a) water washing facilities,
- (b) filtration facilities,
- (c) concentrator (hydrazine),

(d) waste gas washing facilities.

(47) PHARMACEUTICAL MANUFACTURING
  (a) initial preparation facilities,
  (b) filtration facilities,
  (c) separation facilities,
  (d) mixing facilities,
  (e) waste gas washing facilities.

(48) GUNPOWDER MANUFACTURING
  washing facilities

(49) PESTICIDES MANUFACTURING
  mixing facilities

(50) REAGENT MANUFACTURING
  processing facilities

(51) OIL REFINING INDUSTRY
  (a) desalting facilities,
  (b) crude petroleum distilation facilities,
  (c) desulfurisation facilities,
  (d) washing facilities (volatile oil, kerosene or gasoline),
  (e) lubricant washing facilities.

(52) LEATHER MANUFACTURING
  (a) washing facilities,
  (b) lime soaking facilities,
  (c) tannin soaking facilities,
  (d) chrome bathing facilities,
  (e) dyeing facilities.

(53) GLASS MANUFACTURING
  (a) grinding and washing facilities,
  (b) waste gas washing facilities.

(54) CEMENT PRODUCTS MANUFACTURING
  (a) centrifuger,
  (b) moulding facilities,
  (c) wet curing facilities (including steam curing facilities).

(55) READY-MIXED CONCRETE MANUFACTURING
  batcher plant

(56) ORGANIC SAND BOARD MANUFACTURING
  mixing facilities

(57) SYNTHETIC BLACK LEAD ELECTRODE MANUFACTURING
  shaping facilities

(58) RAW POTTERY MATERIALS MANUFACTURING
  (a) water jet crusher,
  (b) separation facilities,
  (c) acid treatment facilities,
  (d) dehydration facilities.

(59) MACADAM QUARRYING
  (a) water jet crusher,
  (b) separation facilities.

(60) GRAVEL QUARRYING
  separation facilities

(61) STEEL INDUSTRY
  (a) tar and gas separation facilities,

- (b) gas cooling and washing facilities,
- (c) rolling facilities,
- (d) hardening facilities,
- (e) wet dust collector

(62) NONFERREOUS METALS MANUFACTURING
- (a) reduction basins,
- (b) electrolysis facilities,
- (c) hardening facilities,
- (d) mercury refinery facilities,
- (e) waste gas washing facilities,
- (f) wet dust collector.

(63) METALIC GOODS MANUFACTURING AND MACHINERY INDUSTRY
- (a) hardening facilities,
- (b) surface treatment facilities (electrolysis)
- (c) cadmium electrode and lead electrode processing facilities,
- (d) mercury refining facilities,
- (e) waste gas washing facilities.

(64) TOWN GAS AND COKE MANUFACTURING
- (a) coal-tar and gas-liquid separating facilities,
- (b) gas cooling and washing facilities (including desulfurisation facilities).

(64)-2 DOMESTIC AND INDUSTRIAL WATER SUPPLY
- (a) sedimentation facilities,
- (b) filtration facilities.

(65) METAL FINISHING INDUSTRY

acid and alkali treatment facilities

(66) ELECTRO-PLATING INDUSTRY

electro-plating facilities

(66)-2 LODGING SERVICE (defined by the provision of paragraph 1 of Article 2 of the Lodging Service Law, excluding boarding house service)
- (a) cooking facilities,
- (b) washing facilities,
- (c) bathing facilities.

(67) LAUNDRY

washing facilities

(68) PHOTO DEVELOPING

automatically developing and washing facilities of film

(69) SLAUGHTER HOUSE

(69)-2 CENTRAL WHOLESALE MARKET OF AQUATIC PRODUCTS
- (a) market yard,
- (b) intermediate market yard.

(70) WASTE OIL TREATMENT FACILITIES

(71) AUTOMATICALLY WASHING FACILITIES OF CAR

(71)-2 RESEARCH, DETERMINATION, MEASUREMENT OR PROFESSIONAL EDUCATION FOR SCIENCE AND TECHNOLOGY (excluding human science)
- (a) washing facilities,
- (b) hardening facilities.

(72) NIGHT SOIL TREATMENT PLANT (more than 501 population served)

(73) SEWAGE TREATMENT PLANT

(74) WASTE WATER TREATMENT PLANT (excluding items 72 and 73)

CHAPTER 2
INSTITUTIONAL STRUCTURES OF WATER POLLUTION CONTROL IN JAPAN

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1. INTRODUCTION

The history of water pollution control in Japan is exceedingly short. The beginning of public hazard caused by water pollution runs back to the "Ashio Copper Mine Pollution Incident" of 1880 and problems had already originated prior to World War II, however, water pollution has gradually expanded since then into various aspects with the development of postwar industries and concentration of population into cities. In 1949, the Public Hazard Ordinance of Tokyo Metropolis was instituted and this gradually spreaded into various cities. In March 1951, a recommendation on water pollution prevention was also made by the Government, however, it did not result in legislation.

In June 10th of 1957, a dispute originated between the fihsermen and the Honshu Paper Mfg. Co., Ltd. (Honshu Paper Mfg. Co. Incident) due to numerous fishes being killed in the Edo river (Tokyo) and with this as the momentum, the Government finally enacted two Laws (Water Quality Preservation Law and Industrial Wastewater Control Law) in September of 1958. However, there existed several defects in these Laws. For example, it was an ex post facto control in which the discharged water quality standard was established by specifying the water body where water pollution originated, compulsory observance of the effluent standard were based on the Industrial Wastewater Control Law and other ten laws, actions taken against violators of the effluent standard were light, etc.

On the other hand, a designated area was established based on Water Quality Preservation Law for rivers as seen in large cities which were polluted by both the industrial wastewater and the domestic sewage and a policy for providing sewerage, etc. by setting the period was formulated. This policy was called the Urban River System and the obligation of discharging favorably treated water by constructing sewage treatment plants within the designated period originated. This was an epochal way of doing for sewerage administration and resultantly, it came to place sewerage in a position of not being merely a facility for improving the living environment but also a facility for controlling the water pollution of public water bodies such as rivers, etc.

The Basic Law Against Public Hazard was enacted in 1967 and the Environmental Standard to be established. Moreover, in 1970, the enactment, alteration and abolition of various Laws related to public hazard were made and the Water Pollution Prevention Law was newly established in place of the conventional Water Quality Preservation Law and the Industrial Wastewater Control Law. Furthermore, the Sewerage Law was also revised for enabling the sewerage to sufficiently serve its role as a water pollution control facility.

By factors such as the establishment of environmental water quality standards based on the Basic Law Against Public Hazard, and designation of the sewage treatment plant as a specified facility under the Water Pollution Prevention Law and its being subjected to the application of the effluent standard, etc., the role and responsibility achieved on water pollution control by sewerage have also been clarified from the point of legislation.

Furthermore, a pollution load control system is to be newly introduced in addition to the conventional concentration control system for conducting a more effective water pollution prevention of closed water bodies such as the Seto Inland Sea, Tokyo Bay, Ise Bay, etc. For this purpose, the Water Pollution Prevention Law was revised in June this year and the provision of sewerage came to be placed in a position where its materialization is the key which controls the success or failure of this system.

Now, we wish to observe mainly from the standpoint of sewerage administration the institutional structures of water pollution control in Japan which has pursued the above-mentioned circumstances and arrived at its condition of today.

2. WATER POLLUTION CONTROL AND SEWERAGE ADMINISTRATION

2.1 BASIC COURSE OF WATER POLLUTION CONTROL

The basic course of water pollution control lies in performing the various policies necessary for achieving and maintaining the environmental water quality standard which has been established based on the Basic Law Against Public Hazard as a goal. Among these policies, the following two constitute the most important elements.

(1) Control of discharged wastewater into public water bodies by the Water Pollution Prevention Law.

(2) Expansion of sewerage and sewage treatment facilities.

Firstly, we will relate on the entire institutional framework of water pollution control and survey the position where sewerage administration is situated within the framework.

Table-2.1 Institutional Structures of Water Pollution Control

Law	Contents of Water Pollution Control			Responsible Person
Basic Law Against Public Hazard	Environmental Water Quality Standard	Environmental standard concerning human health		National government
		Environmental standard on living environment		National government
			Designation of the type of water body	Prefectural Governor (National government is responsible for interprefectural bodies)
	Public Hazard Prevention Plan	Instruction of basic policy		Prime Minister →Governor
		Preparation of plan		Governor
		Approval of plan		Prime Minister
Water Pollution Prevention Law	Control of discharged wastewater	Effluent standard	Uniform standard	National government
			Severer standard	Prefectural government
			Additional standard	Prefectural and local government
		Pollution load control	Basic policy for pollution load reduction	Prime Minister
			Plan for pollution load reduction	Governor
			Standard for pollution load control	Governor
		Supervision of factories, etc.	Notification on establishment and changes of specified facilities	Dischargers →Governor
			Alteration order of plan	Governor
			Improvement order	Governor
			Report and inspection	Governor
	Supervision of water pollution condition	Constant supervision		Governor
		Preparation of measuring plan		Governor
Sewerage Law	Comprehensive Basin-wide Sewerage Study Program	Preparation of plan		Governor
		Approval of plan		Minister of Construction (Consult with the Minister of the Environment Agency)
	Construction and management of sewerage	Public sewerage		Municipalities
		Regional sewerage		Prefectural government
	Compulsory use of public sewerage	Obligation for providing drainage equipment		Users, etc.
		Obligation for remodelling into a flush toilet		Users
	Control of industrial wastewater	Notification on establishment and changes of specified facilities		To sewerage superintendent
		Alteration order of plan		Sewerage superintendent
		Obligation for equipping pretreatment facilities		Users
		Improvement order		Sewerage superintendent
		Report and inspection		Sewerage superintendent

Other Related Laws:

○ Ocean Pollution Prevention Law ○ River Law ○ Mining Safety Law ○ Natural Park Law ○ Coal Washing Industry Law ○ Harbor Regulations Law ○ Marine Resources Protection Law ○ Scraps and Wastes Disposal & Cleaning Law ○ Poisonous Substances and Powerful Drugs Control Law ○ Agricultural Medicines Control Law ○ Pollution Prevention Law of Land for Agricultural Use ○ Public Hazard Disputes Disposition Law ○ Law for Compensating Health Harms by Public Hazard ○ Law on Enterpriser's Burden for Public Hazard Prevention Expenses ○ Public Hazard Prevention Agency Law ○ Public Hazard Crime Punishment Law

2.2 BASIC LAW AGAINST PUBLIC HAZARD

The Basic Law Against Public Hazard has been enacted for the purpose of protecting the health of the people and preserving the integrity of living environments, and together with clarifying the obligations of the enterpriser, national and local government on public hazard prevention, this law is for plotting the comprehensive promotion of public hazard policy by establishing matters which become basis of policies on public hazard prevention (Article 1 of the Law).

2.2.1 Definition of Public Hazard

Public hazard referred to the Law means the origination of harms affecting the health of people or living environments by air pollution, water pollution, soil pollution, noises, vibrations, sinking of ground and odors which are caused by industrial activities or other activities of people (Article 2 of the Law).

2.2.2 Basic Policies on Public Hazard Prevention

a. Environmental Standard

In regards to the environmental standard on air pollution, water pollution, soil pollution and noises, it has been decided to establish a standard which is desirable for maintaining in protecting the health of the people and preserving the integrity of the living environment (Article 9 of the Law).

The Environmental Water Quality Standard was decided upon by the Cabinet in April 21, 1971 and its notification issued in December 26, 1971. In this Standard, there are both the environmental standard on the protection of people's health and the environmental standard on the preservation of integrity of living environments (See Tables 2.2 and 2.3). In the former standard, an uniform standard is established for all public water bodies on health affecting items such as cadmium, mercury, etc. In the latter standard, a standard is established for each type of public water body on environmental items such as pH, BOD, etc., and the designation of the type of water bodies to which each public water body corresponds is made by the Minister of the Environment Agency (With the exception of interprefectural public water bodies, entrusted to prefectural governors).

Table-2.2 Environmental Standard Concerning People's Health

Item	Cyanide	Alkyl-mercury	Organic phosphorus	Cadmium	Lead	Sexi-valent chrome	Arse-nic	Total mercury	PCB
Standard value	Not to be detected	Not to be detected	Not to be detected	Less than 0.01 ppm	Less than 0.1 ppm	Less than 0.05 ppm	Less than 0.05 ppm	Less than 0.0005 ppm	Not to be detected

Table-2.3 Environmental Standard Concerning Living Environment

(1) Rivers

Item / Type classification	Adaptability of purpose of use	Standard Value				
		Hydrogen ion concentration (pH)	Biochemical oxygen demand (BOD)	Suspended solids (SS)	Dissolved oxygen (DO)	No. of coliform group
AA	First class waterworks Preservation of natural environment and those listed in columns below A	More than 6.5 and less than 8.5	Less than 1 ppm	Less than 25 ppm	More than 7.5 ppm	Less than 50MPN/ 100ml
A	Second class waterworks First class fisheries Bathing and those listed in column below B	More than 6.5 Less than 8.5	Less than 2 ppm	Less than 25 ppm	More than 7.5 ppm	Less than 1,000 MPN/100 ml
B	Third class waterworks Second class fisheries and those listed in columns below C	More than 6.5 and less than 8.5	Less than 3 ppm	Less than 25 ppm	More than 5 ppm	Less than 5,000 MPN/100 ml
C	Third class fisheries First class industrial water and those listed in columns below D	More than 6.5 and less than 8.5	Less than 5 ppm	Less than 50 ppm	More than 2 ppm	–
D	Second class industrial water Agricultural water and those listed in column E	More than 6.0 and less than 8.5	Less than 8 ppm	Less than 100 ppm	More than 2 ppm	–
E	Third class industrial water Environmental preservation	More than 6.0 and less than 8.5	Less than 10 ppm	Suspension of dusts, etc., not to be detected	More than 2 ppm	–

(Notes):

1. Preservation of natural environment : Preservation of environments of natural sight-seeing sceneries, etc.
2. First class waterworks : Those performing simple purification operations by means of filtration, etc.
   Second class waterworks Those performing normal purification operations of settling and filtration, etc.
   Third class waterworks : Those performing high-degree purification operations which accompany pre-treatment, etc.
3. First class fisheries : Those for marine creatures in water body of oligosaprobien such as trout and char, and those for marine creatures of Second and Third class fisheries.
   Second class fisheries : Those for marine creatures in water body of oligosaprobien such as the Oncorhynchus genus, sweetfish, etc., and those for marine creatures of Third class fisheries.
   Third class fisheries : Those for marine creatures in water basin of β-mesosaprobiens such as carp, roach, etc.
4. First class industrial water : Those performing normal purification operations by sedimentation, etc.
   Second class industrial water : Those performing high-degree purification operations by adding chemicals, etc.
   Third class industrial water : Those performing special purification operations.
5. Environment preservation : Limit which does not originate an unpleasant impression in the daily life of the people (including strolls at coast).

(2) Lakes & Marshes (Natural lakes & marshes and artificial lakes with a reservoir capacity of more than 10 million cubic meters

Item / Type classification	Adaptability of purpose use	Standard Value				
		Hydrogen ion concentration (pH)	Chemical oxygen demand (COD)	Suspended solids (SS)	Dissolved oxygen (DO)	No. of coliform group
AA	First class waterworks, First class fisheries Natural environment preservation and those listed in columns below A	More than 6.5 and less than 8.5	Less than 1 ppm	Less than 1 ppm	More than 7.5 ppm	Less than 50MPN/100ml
A	Second and third class waterworks Second class fisheries Bathing and those listed in columns below B	More than 6.5 and 8.5	Less than 3 ppm	Less than 5 ppm	More than 7.5 ppm	Less than 1,000 MPN/100 ml
B	Third class fisheries First class industrial water Agricultural water and those listed in column C	More than 6.5 and less than 8.5	Less than 5 ppm	Less than 15 ppm	More than 5 ppm	–
C	Second class industrial water Environmental preservation	More than 6.0 and less than 8.5	Less than 8 ppm	Suspension of dusts, etc., not to be detected	More than 2 ppm	–

(Notes):
1. Natural environment preservation : Preservation of environments of natural sight-seeing sceneries, etc.
2. First class waterworks : Those performing simple purification operations by means of filtration, etc.
Sedond and third class waterworks : Those performing normal purification operations by settling and filtration, etc., or those performing high-degree purification operations which accompany pre-treatment, etc.
3. First class fisheries : Those for marine creatures in water basin of poor nutrition type lakes such as trout, etc., and those for marine creatures of second and third class fisheries.
Second class fisheries : Those for marine creatures in water basin of poor nutrition type lakes such as the Oncorhynchus genus, sweetfish, etc., and those for marine creatures of third class fisheries.
Third class fisheries : Those for marine creatures such as carp, char, etc., in water basin of rich nutrition type lakes.
4. First class industrial water : Those performing normal purification operations by sedimentation, etc.
Second class industrial water : Those performing high-degree purification operations by adding chemicals,etc., or those performing special purification operations.
5. Environmental preservation : Limit which does not originate an unpleasant impression in the daily life of people (including strolls at coast).

(3) Seas

Item / Type classification	Adaptability of purpose use	Standard Value				
		Hydrogen ion concentration (pH)	Chemical oxygen demand (COD)	Dissolved oxygen (DO)	No. of coliform group	N-hexane extraction (Oily matters)
A	First class fisheries, Bathing and those listed in columns below B	More than 7.8 and less than 8.3	Less than 2 ppm	More than 7.5 ppm	Less than 1,000 MPN/100ml	Not to be detected
B	Second class fisheries, Industrial water and those listed in column C	More than 7.8 and less than 8.3	Less than 3 ppm	More than 5 ppm	–	Not to be detected
C	Environmental preservation	More than 7.0 and less than 8.3	Less than 8 ppm	More than 2 ppm	–	–

(Notes): 1. First class fisheries : Those for marine creatures such as red sea-bream, yellow-tail, Wakame seaweed, etc., and those for marine creatures of second class fisheries.

Second class fisheries : Those for marine creatures such as gray mullet, laver, etc.

2. Environmental preservation Limit which does not originate an unpleasant impression in the daily life of people (including strolls at coast).

b. Policies for Achieving and Maintaining the Environmental Standard

The following policies to be performed by the national and local government are established.

Control on discharged wastewater, etc. (Article 10 of the Law), Control on land use and establishment of facilities (Article 11), Promotion of providing facilities (sewerage, etc.) on public hazard prevention (Article 12), Consolidation of supervision and measurement setup (Article 13), Execution of investigations (Article 14), Furtherance of scientific techniques (Article 15), Diffusion of knowledge (Article 16), and Considerations of public hazard prevention in regional development policies and protection of natural environment (Section 2 of Article 17).

c. Public Hazard Prevention Plan

The Public Hazard Prevention Plan is a plan concerning policies for public hazard prevention which is carried out systematically and collectively in specific areas where public hazards are concentrated. Areas where the public hazard prevention plan is schemed are areas where public hazards are presently extreme or where there lies a fear of public hazards becoming conspicuous due to the rapid concentration of population and industries and unless policies on public hazard prevention are collectively adopted, it would become extremely difficult to attain public hazard prevention. The proceedings on plan scheming are that the Prime Minister decides the area, indicates the basic policy and instructs the scheming of the plan to the concerned Governor after listening to the opinions of the prefectural Governor, and then the concerned Governor prepares the plan and receives the approval of the Prime Minister (Article 19 of the Law).

In the Public Hazard Prevention Plan, the goal is placed on environmental standard, the discharge control methods necessary for achieving the environmental standard within the stipulated period of 5 – 10 years are indicated and the required amount of public hazard elimination facilities such as sewerage, etc. is estimated as the bases of the plan. On water pollution, it is impossible to attain the target of the plan by merely controlling the discharges of wastewater as the pollution load discharged from general households also occupies a considerable percentage. Thus, the construction of a considerable amount of sewerage facilities is required. The total amount of necessary public investments is listed in the Public Hazard Prevention Plan, however, more than 70% of the total amount is for sewerage construction expenses.

Table-2.4 Actual Conditions of Public Hazard Prevention Plan

Area classification	Name of area	Term for accomplishment	Expenses for public hazard policy (Unit: 100 million yen)	Plan for sewerage	
				Served population (Unit: 1000 persons)	Served area (ha)
First area	Chiba. Ichihara area	To be included in Chiba littoral districts and reconsidered			
	Yokkaichi area	1977 (4 years)	230	69	1,093
	Mizushima area		220	106	662
Second area	Tokyo area	1981 (5 years)	22,290	8,661	66,047
	Kanagawa area	1981 (5 years)	11,120	3,380	29,885
	Osaka area	1981 (5 years)	11,520	5,581	42,826
	Chiba Prefecture, Edo River drainage basin	To be included in Chiba littoral districts and reconsidered			
	Saitama Prefecture, Ara River system drainage basin	1981 (5 years)	4,390	1,622	25,433
	Kyoto Prefecture, Yodo River drainage basin	1981 (5 years)	3,580	1,213	10,074
	Nara Prefecture, Yamato River drainage basin	1981 (5 years)	1,750	342	4,846
Third area	Kashima area	1981 (5 years)	190	18	2,250
	Nagoya, etc., area	1981 (5 years)	4,290	2,040	23,196
	Hyogo Prefecture, Eastern area	1981 (5 years)	3,380	1,433	13,118
	Kita-Kyushu area	1981 (5 years)	1,830	820	8,800
	Oita area	1981 (5 years)	400	142	1,305
Fourth area	Fuji area	1977 (5 years)	190	154	1,136
	Harima southern area	1977 (5 years)	1,100	544	5,836
	Otake area	1977 (5 years)	140	37	470
	Iwakuni area	1977 (5 years)	130	44	473
	Omuta area	1977 (5 years)	130	32	373
	Saitama area	1984 (9 years)	(340)	–	–

Table-2.4 Actual Conditions of Public Hazard Prevention Plan (Continued)

Area classification	Name of area	Term for accomplishment	Expenses for public hazard policy (Unit: 100 million yen)	Plan for sewerage	
				Served population (Unit: 1000 persons	Served area (ha)
Fifth area	Tomakomai area	1978 (5 years)	170	140	1,751
	Sendai Bay area	1978 (5 years)	810	588	7,507
	Iwaki area	1978 (5 years)	820	124	1,595
	Chiba littoral districts	1978 (5 years)	3,500	1,460	14,196
	Toyama, Takaoka area	1978 (5 years)	450	211	2,170
	Koromoura vicinity area	1978 (5 years)	1,430	319	11,664
	Kobe area	1978 (5 years)	1,550	1,552	10,780
	Bingo area	1978 (5 years)	830	336	3,477
	Shunan area	1978 (5 years)	490	164	1,897
	Toyo area	1978 (5 years)	290	79	1,378
Sixth area	Muroran area	1979 (5 years)	120	114	1,054
	Hachinohe area	1979 (5 years)	230	20	273
	Niigata area	1979 (5 years)	440	142	1,397
	Shizuoka, Shimizu area	1979 (5 years)	223	200	1,232
	Hiroshima, Kure area	1979 (5 years)	990	394	3,148
	Shimonoseki, Ube area	1979 (5 years)	280	167	1,550
	Kagawa area	1979 (5 years)	370	182	1,925
	Wakayama area	1979 (5 years)	290	45	530
	Okayama, Bizen area	1979 (5 years)	370	200	1,849
Seventh area	Sapporo area	1980 (5 years)	2,050	1,303	16,285
	Akita area		390	190	766
	Hitachi area		180	94	1,397
	Matsumoto, Suwa area		320	190	830
	East Mikawa area		770	434	3,209
	Gifu, Ogaki area	1980 (5 years)	550	497	3,406
	East Nomi area		120	44	331
	Tokushima area		220	87	697
	Hiyuga, Nobeoka area		70	63	429

Note): The expenses for the Public Hazard Policy is the total expenses which includes the sewerage construction costs.

Furthermore, on public hazard prevention projects which are based on this plan, a subsidy of a higher percentage than same projects in other areas is generally made pursuant to the "Law Concerning the Special Financial Measures by National Government for Projects on Public Hazard Prevention".

2.3 WATER POLLUTION PREVENTION LAW

The Water Pollution Prevention Law contrives to prevent public water bodies from pollution by controlling discharged wastewater from factories and the like and preserves the living environment as well as protecting the health of people, and aims at promoting the protection of victims by providing the responsibility of the enterpriser's indemnification for damages caused in the case that damages originate on the health of the people by wastewater which are discharged from factories and the like (Article 1 of Water Pollution Prevention Law).

The control of discharged wastewater under the Water Pollution Prevention Law plots the water pollution control of public water bodies by making the factories and the like to be provided with specified facilities in discharging wastewater into public water bodies report on their provision of such specified facilities and whenever necessary, together with issuing a plan alteration order and having the wastewater treatment setup consolidated, and assigning the obligation of observing the effluent standard and applying the penal regulations when there lies a fear of violating the effluent standard.

2.3.1 Object of Discharged Wastewater Control

The "Discharged Wastewater" which becomes the object of control under the Water Pollution Prevention Law refers to the water discharged into public water bodies from factories or the like (specified works) which are provided with specified facilities (Article 2, Section 3 of Water Pollution Prevention Law).

"Specified Facilities" refers to facilities discharging wastewaters which contain substances such as cadmium and the like affecting health or facilities discharging wastewater to the extent where there lies a fear of damages affecting living environments originating on items indicating the water pollution condition such as pH, BOD, etc., and it is concretely specified in the Government Ordinance. A sewage treatment facility also corresponds to the specified facilities.

2.3.2 Effluent Standard

The effluent standard is the tolerance limit stipulated for each item on the polluted condition of the discharged wastewater and is classified into the uniform standard and the severer standard.

a. Uniform Standard

The uniform standard is decided by the Ordinance of the Prime Minister's Office and there are the "General Standard" and the "Provisional Standard". The General Standard is a standard which is applied uniformly for all public water bodies and for all specified works, however, the Provisional Standard is a tentative and lenient standard which is applicable for the restricted 5 years period from June 24, 1976 in place of the General Standard of environmental items and is provided for several specific industrial classifications.

Table-2.5 Uniform Standard

(1) Items Concerning Health Protection of People

Item	Cyanide	Alkyl-mercury	Organic phosphorus	Cadmium	Lead	Sexivalent chrome	Arsenic	Total mercury	PCB
Standard value	Max. 1 ppm	Not to be detected	Max. 1 ppm	Max. 0.1 ppm	Max. 1 ppm	Max. 0.5 ppm	Max. 0.5 ppm	Max. 0.005 ppm	Max. 0.003 ppm

(2) Items Concerning Preservation of Living Environments (General Items)

Item	pH		BOD		COD		SS	
Standard value	Rivers, lakes & marshes	Seas	Average daily	Max.	Average daily	Max.	Average daily	Max.
	5.8 – 8.6	5 – 9	120 ppm	160 ppm	120 ppm	160 ppm	150 ppm	200 ppm

Remarks:

1. This standard shall be applied to factories or the like of generally more than $50m^3$ of discharged wastewater per day.
2. The standard of COD shall not be applicable to factories or the like discharging water into rivers and the standard of BOD shall not be applicable to those discharging water into lakes, marshes and seas.
3. The standard of pH shall not be applicable for the sulphur mining industry (including pyrrhotite mining industry which coexists with sulphur).

(3) Items Concerning Preservation of Living Environments (Special Items)

Item	Oily matters (n-hexane extraction)		Phenol	Copper	Zinc	Iron (Soluble)	Manganese (Soluble)	Chrome	Fluorine	No. of coliform group
	Petroleum group oily matter	Animal & plant oils and fats, etc.								
Standard value	Max.	Max.	Max.	Max.	Max.	Max.	Max.	Max.	Max.	Average daily
	5 ppm	30 ppm	5 ppm	3 ppm	5 ppm	10 ppm	10 ppm	2 ppm	15 ppm	3000/cm^3

Remarks:

1. This standard shall be applied to factories or the like of generally more than $50m^3$ of discharged wastewater per day.
2. The standard on iron (soluble) shall not be applicable for the sulphur mining industry (including pyrrhotite mining industry which coexists with sulphur).

b. Severer Standard

In case that a section which can be recognized as insufficient for protecting the health of people or for preserving the living environments under the uniform standard exists in public water water bodies within the territory of prefecture concerned, judging from natural and social conditions, the competent prefecture may stipulate by Ordinance a severer effluent standard up to the extent that is required and sufficient for maintaining the environmental water quality standard, for the discharged wastewater in that section (Article 3 Section 3 of Water Pollution Prevention Law). This standard is called the "Severer Standard".

Since the Severer Standard stipulates a tolerance limit which is severer than the tolerance limit stipulated in the Uniform Standard, its objective substances and items shall be limited to substances and items that are the objects under the Uniform Standard. Discharge control on others besides the objective substances and items under the Uniform Standard (Additional Control) and discharge control of discharged wastewater from factories or the like other than the specified works (Extended Control) can be stipulated by the Ordinance of local government according to Article 29 of the Water Pollution Prevention Law.

2.4 SEWERAGE LAW

The Sewerage Law stipulates the matters concerning the Comprehensive Basin-wide Sewerage Study Program and standards for the establishement and management of public sewerage, regional sewerage and urban storm water channel for promoting sewerage works, and aims at contributing to the sound development and improvement of public health of cities and at the same time, contributing to the water pollution control of public water bodies (Article 1 of Sewerage Law).

2.4.1 Comprehensive Basin-wide Sewerage Study Program

Among public water bodies with a environmental standard, for water bodies which produce water pollution by wastewater from more than two municipalities districts and which can be considered necessary to achieve the environmental water quality standard by mainly providing sewerage, the prefectural government must prepare the Comprehensive Basin-wide Sewerage Study Program (Article 2, Item 2 of Sewerage Law).

In this Program, stipulations must be made for the basic policy for providing sewerage, the area where the sewage should be collected and treated, arrangement, structure and capacity of basic facilities of sewerage and the executing ranking of sewerage projects, by taking into consideration the natural conditions of the competent area, outlook on quantity and quality of wastewater, cost effectiveness analysis of expenditures for sewerage, etc. The Comprehensive Basin-wide Sewerage Study Program is so called an integrated basic plan for sewerage systems and individual sewerage project shall be carried out in conformity with this Program (Article 6, No. 5 and Article 25, Item 5, No. 4 of Sewerage Law).

2.4.2 Use of Public Sewerage

When the use of public sewerage is commenced, the landowner (building owner in case of building sites, public facility manager in case of public facility sites) within the drainage district must promptly provide necessary drainage equipment for allowing the sewage of the land to flow into the public sewerage. However, even though he may be the responsible person for providing drainage equipment, connections to public sewerage do not have to be made, upon receiving the permission by the superintendent of public sewerage when the cooling water, or the like of the factory conforms to the effluent standard to public water bodies and is approved that the direct discharge to public water bodies is rational (Article 10, Section 1 of Sewerage Law).

When sewage treatment is commenced by the sewage treatment plant of public sewerage or by the sewage treatment plant of regional sewerage to which the competent public sewerage connects, the owner of the building where a non-flush toilet is provided within the treatment district must remodel the toilet into a flush toilet within three years from the day of treatment commencement (Article 11, Item 3, Section 1 of Sewerage Law).

2.4.3 Management of Public Sewerage

a. Water Quality Control of Effluent

It is necessary to appropriately perform water quality control of effluent from sewerage in order for the sewerage to contribute to the water pollution control of public water bodies. For this purpose, it is defined in the Sewerage Law that the quality of effluent from public sewerage or regional sewerage into rivers and other public water bodies must conform to the technical standards stipulated in Government Ordinance (Article 8 of Sewerage Law).

Article 6, Section 1 of the Ordinance stipulates the water quality standard of effluent for each sewage treatment method (See Table 2.6 below).

Table-2.6 Technical Standard of Water Quality of Effluent

Classification	pH	BOD (mg/l)	SS (mg/l)	No. of coli-form group
Activated sludge process Low-rate trickling filter process or the like	5.8 – 8.6	Less than 20	Less than 70	Less than 3,000
High-rate trickling filter process Modified aeration or the like	5.8 – 8.6	Less than 60	Less than 120	Less than 3,000
Sedimentation	5.8 – 8.6	Less than 120	Less than 150	Less than 3,000

Moreover, in Section 2 of Article 6 of the Ordinance, when the effluent standard based on the stipulation of Article 3 of the Water Pollution Prevention Law (to include severer standard as well as uniform standard) is severer than this water quality standard or when the additional standard is stipulated by ordinance of local government, it is made to treat the effluent standard as the standard of effluent based on the Sewerage Law.

Generally speaking it can be said that a severer standard than the standard of the Water Pollution Prevention Law is spontaneously placed on BOD.

b. Control on Dischargers of Hazardous Wastewater

(i) Control of Discharged Wastewater from Specified Works

Those who discharge wastewater from factories or the like (Special Works) which are provided with specified facilities under the Water Pollution Prevention Law and use public sewerage must not discharge wastewater which do not conform to a fixed water quality standard (2 of Article 12 of Sewerage Law). Panel regulations shall be applicable to violators (2 of Article 46 of Sewerage Law).

As to the fixed water quality standard, an uniform standard is stipulated (same as the uniform standard of the Water Pollution Prevention Law) by Government Ordinance for those difficult to treat at sewage treatment plants such as cadmium, etc., as health items and zinc, etc., as environment items, and as to other items such as BOD, etc. which are possible of treating at sewage treatment plants, stipulations are respectively made by municipal ordinances according to the standards stipulated by the Government Ordinance.

The standard values for the former health itmes, etc. are the same values as in the case of direct discharge into public water bodies. Therefore, specified works will be subjected to an entirely same control on these items as in the case of discharge into public water bodies.

However, it does not mean that all wastewaters from the specified works will become objects of immediate punishment, but small amount of wastewater is excluded. As for the wastewater which is not subjected to immediate punishment but contains a fear of making water quality of effluent from the sewerage extremely difficult to conform to the technical standard of Article 8 of the Sewerage Law, the superintendent of public sewerage can obligate the provision of pretreatment facilities by ordinance (10 of Article 12 of Sewerage Law). Moreover, the provision of pretreatment facilities can be obligated by ordinance to those who use public sewerage by continuously discharging wastewater having the fear of extremely obstructing the functions of sewerage facilities or damaging facilities (Article 12 of Sewerage Law).

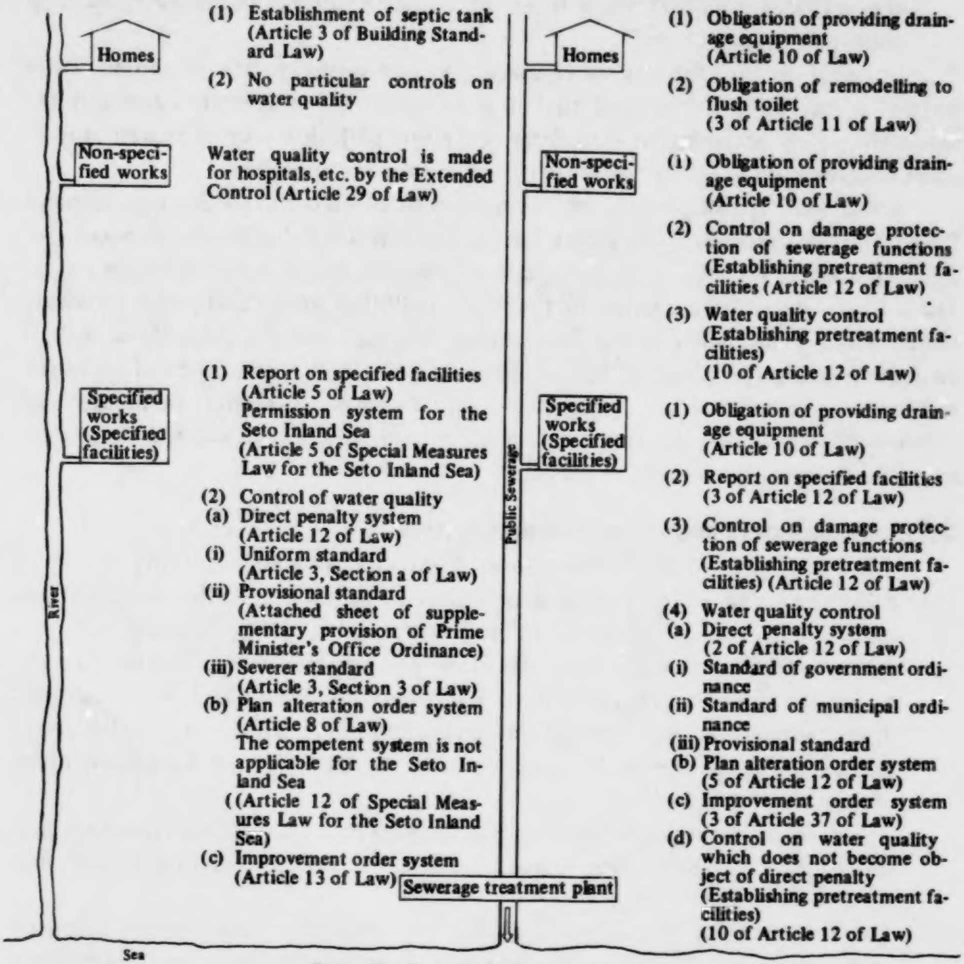
Besides the above, the pre-checking system which requests reporting on establishment of specified facilities and issues a alteration order of plan whenever necessary to specified works that uses public sewerage by discharging wastewater, and the improvement order system which orders improvement of structures of specified facilities in case that it is recognized that a fear exists of discharging wastewater which violates the water quality standard are stipulated.

(ii) Control of Discharged Wastewater from Non-Specified Works

A direct penalty is not imposed on discharged wastewater from works other than the specified works. However, when the wastewater with a fear of extremely obstructing the functions of sewerage facilities or damaging facilities is discharged, or when the wastewater with a fear of making effluent from sewerage extremely difficult to conform to the technical standard of Article 8 of the Sewerage Law is discharged, establishment of pretreatment facilities can be obligated by ordinance (Article 12, 10 of Article 12 of Sewerage Law).

The controls by both Water Pollution Prevention Law and Sewerage Law are compared and illustrated, as shown in Fig. 2.1.

Fig.-2.1 Comparison of Controls by Both Water Pollution Prevention Law and Sewerage Law



When discharged to public water bodies
(Water Pollution Prevention Law)

(1) Establishment of septic tank (Article 3 of Building Standard Law)
(2) No particular controls on water quality

Water quality control is made for hospitals, etc. by the Extended Control (Article 29 of Law)

(1) Report on specified facilities (Article 5 of Law)
Permission system for the Seto Inland Sea (Article 5 of Special Measures Law for the Seto Inland Sea)
(2) Control of water quality
(a) Direct penalty system (Article 12 of Law)
(i) Uniform standard (Article 3, Section a of Law)
(ii) Provisional standard (Attached sheet of supplementary provision of Prime Minister's Office Ordinance)
(iii) Severer standard (Article 3, Section 3 of Law)
(b) Plan alteration order system (Article 8 of Law)
The competent system is not applicable for the Seto Inland Sea
((Article 12 of Special Measures Law for the Seto Inland Sea)
(c) Improvement order system (Article 13 of Law)

When discharged to sewerage
(Sewerage Law)

(1) Obligation of providing drainage equipment (Article 10 of Law)
(2) Obligation of remodelling to flush toilet (3 of Article 11 of Law)

(1) Obligation of providing drainage equipment (Article 10 of Law)
(2) Control on damage protection of sewerage functions (Establishing pretreatment facilities (Article 12 of Law)
(3) Water quality control (Establishing pretreatment facilities) (10 of Article 12 of Law)

(1) Obligation of providing drainage equipment (Article 10 of Law)
(2) Report on specified facilities (3 of Article 12 of Law)
(3) Control on damage protection of sewerage functions (Establishing pretreatment facilities) (Article 12 of Law)
(4) Water quality control
(a) Direct penalty system (2 of Article 12 of Law)
(i) Standard of government ordinance
(ii) Standard of municipal ordinance
(iii) Provisional standard
(b) Plan alteration order system (5 of Article 12 of Law)
(c) Improvement order system (3 of Article 37 of Law)
(d) Control on water quality which does not become object of direct penalty (Establishing pretreatment facilities) (10 of Article 12 of Law)

Control by Water Pollution Prevention Law

2.5 POLLUTION LOAD CONTROL SYSTEM

2.5.1 Background of Introducing Pollution Load Control System

The recent water pollution conditions of public water bodies in Japan are relatively in the trend of improvement, however, water pollution is particularly conspicuous in widely closed water bodies such as Tokyo Bay, Ise Bay, Seto Inland Sea, etc. where numerous pollution generating sources concentrate. Thus, the improvement of water quality in these water bodies has become an urgent problem.

In December last year, a report by the Central Council for Public Hazard Policy has been made on "the way of pollution load control system" and the following has been stated in the report.

> "It is essential to reduce in overall the pollution load which affects the water quality of water bodies. For this purpose, together with making all industrial

wastewater and domestic sewage as objects, it is necessary to direct attention to the establishment of a comprehensive water pollution control policy which also includes pollution loads from the so-called internal production and non-point pollution sources."

In other words, for the water quality improvement policy of closed water bodies, it has been emphasized that it is necessary to collectively carry out the reduction policy of pollution load from non-point pollution sources as well as from point sources and the policy for nutritive salts in the water bodies.

Amid such a background, the "Amendment of Seto Inland Sea Environment Preservation Extraordinary Measures Law and the Water Pollution Prevention Law" was promulgated on June 13 this year and is subjected to enforcement within a year. These Laws constitute a portion of the water pollution prevention policy in widely closed water bodies such as the Seto Inland Sea, etc., and the contents consist of institutionalizing the pollution load control on point pollution sources of industrial wastewaters, sewerages, etc. which flow into these water bodies, preparation of recommended guideline for reducing designated substances such as phosphorus, etc. in the Seto Inland Sea and other matters.

2.5.2 Outline of Pollution Load Control System

The main points of the Pollution Load Control System are as follows.

(1) For preventing water pollution in widely closed water bodies recognized as being difficult to maintain the environmental water quality standard under the present effluent standard only, the Prime Minister shall decide the basic policy on pollution load reduction (Basic Policy for Pollution Load Reduction) on districts related to water pollution (designated districts) of water bodies specified according to chemical oxygen demand (COD) and other designated items (designated water bodies).

(2) The Prefectural Governor must decide the plan for achieving the reduction goal (Pollution Load Reduction Plan) based on the above-mentioned Basic Policy for Pollution Load Reduction.

(3) The Prefectural Governor must decide the Pollution Load Control Standard based on the Pollution Load Reduction Plan for the discharged wastewater from a specified works which is above a fixed scale within the designated district (works within designated district).

(4) The Prefectural Governor shall be able to order measures for improvement, etc. of wastewater treatment methods when a fear exists of discharged wastewater not conforming to the Pollution Load Control Standard.

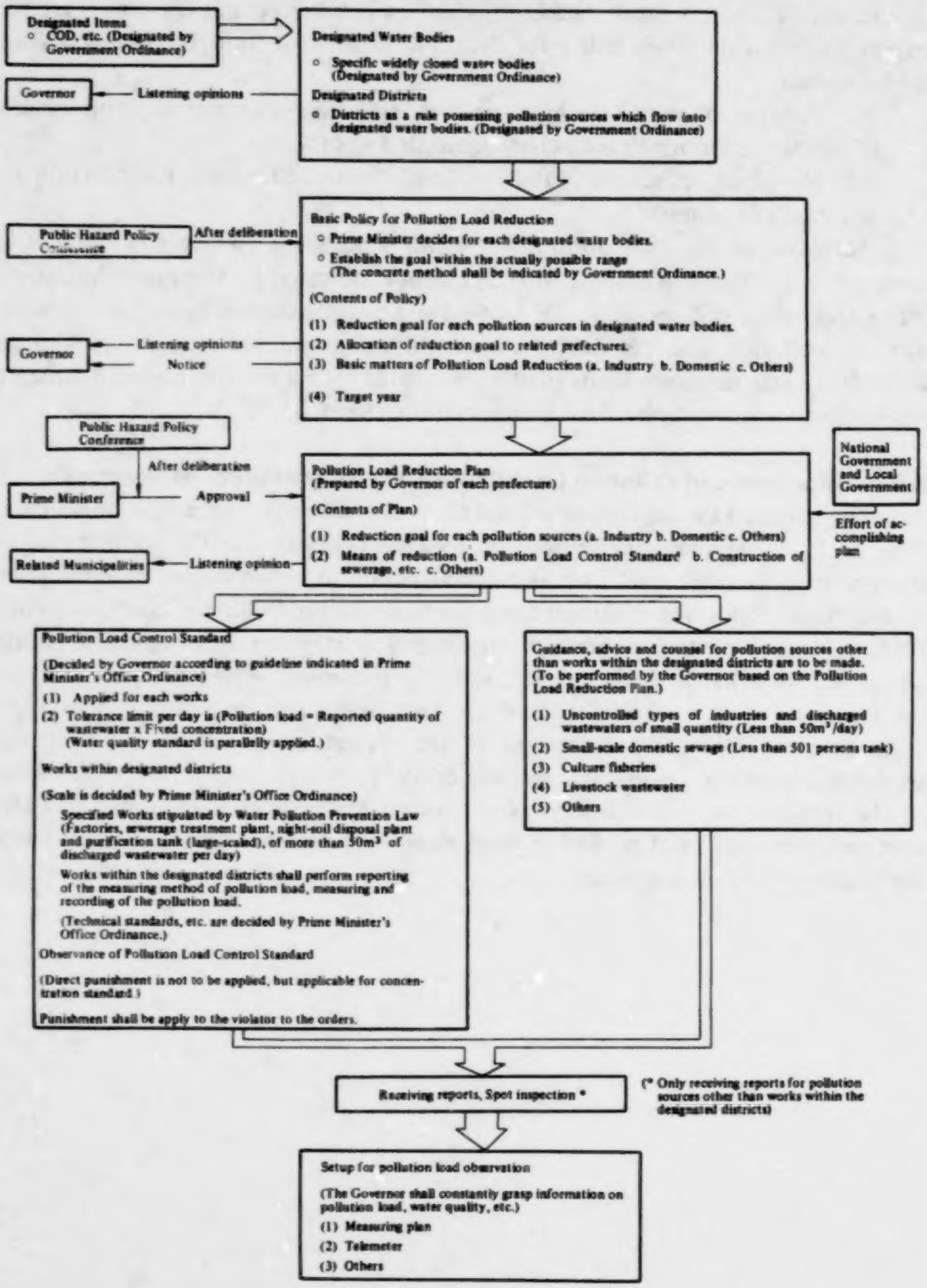
(5) The Prefectural Governor shall be able to give guidance, advice and counsel which are necessary for reducing the pollution load to dischargers other than "the works within designated district" for accomplishing the Pollution Load Reduction Plan.

(6) Persons discharging water from "the works within designated district" must measure the pollution load and record the results.

The above is the gist of the law revision on the Pollution Load Control System, and the Seto Inland Sea, Tokyo Bay and Ise Bay are expected to be the "designated water bodies" which are subjected to pollution load control. The "designated item" which becomes the object of control is only COD for the time being.

In the "Basic Policy for Pollution Load Reduction", the following are to be decided.

Fig.-2.2 Flowchart of Pollution Load Control



Designated Items
○ COD, etc. (Designated by Government Ordinance)
Governor
Listening opinions
Designated Water Bodies
○ Specific widely closed water bodies (Designated by Government Ordinance)
Designated Districts
○ Districts as a rule possessing pollution sources which flow into designated water bodies. (Designated by Government Ordinance)
Public Hazard Policy Conference
After deliberation
Basic Policy for Pollution Load Reduction
○ Prime Minister decides for each designated water bodies.
○ Establish the goal within the actually possible range (The concrete method shall be indicated by Government Ordinance.)
(Contents of Policy)
(1) Reduction goal for each pollution sources in designated water bodies.
(2) Allocation of reduction goal to related prefectures.
(3) Basic matters of Pollution Load Reduction (a. Industry b. Domestic c. Others)
(4) Target year
Governor
Listening opinions
Notice
Public Hazard Policy Conference
After deliberation
Prime Minister
Approval
Pollution Load Reduction Plan (Prepared by Governor of each prefecture)
(Contents of Plan)
(1) Reduction goal for each pollution sources (a. Industry b. Domestic c. Others)
(2) Means of reduction (a. Pollution Load Control Standard b. Construction of sewerage, etc. c. Others)
National Government and Local Government
Effort of accomplishing plan
Related Municipalities
Listening opinion
Pollution Load Control Standard
(Decided by Governor according to guideline indicated in Prime Minister's Office Ordinance)
(1) Applied for each works
(2) Tolerance limit per day is (Pollution load = Reported quantity of wastewater x Fixed concentration) (Water quality standard is parallelly applied.)
Works within designated districts
(Scale is decided by Prime Minister's Office Ordinance)
Specified Works stipulated by Water Pollution Prevention Law (Factories, sewerage treatment plant, night-soil disposal plant and purification tank (large-scaled), of more than 50m³ of discharged wastewater per day)
Works within the designated districts shall perform reporting of the measuring method of pollution load, measuring and recording of the pollution load.
(Technical standards, etc. are decided by Prime Minister's Office Ordinance.)
Observance of Pollution Load Control Standard
(Direct punishment is not to be applied, but applicable for concentration standard.)
Punishment shall be apply to the violator to the orders.
Guidance, advice and counsel for pollution sources other than works within the designated districts are to be made. (To be performed by the Governor based on the Pollution Load Reduction Plan.)
(1) Uncontrolled types of industries and discharged wastewaters of small quantity (Less than 50m³/day)
(2) Small-scale domestic sewage (Less than 501 persons tank)
(3) Culture fisheries
(4) Livestock wastewater
(5) Others
Receiving reports, Spot inspection *
(* Only receiving reports for pollution sources other than works within the designated districts)
Setup for pollution load observation
(The Governor shall constantly grasp information on pollution load, water quality, etc.)
(1) Measuring plan
(2) Telemeter
(3) Others

(1) Amount of reduction goal for each pollution source in designated water bodies.

(2) Allocation of the amount of reduction goal to related prefectures.

(3) Basic matters of pollution load reduction. (Classified according to industrial, domestic and others)

(4) Target year: 1984 (Intermediate target year: 1981)

In the "Pollution Load Reduction Plan" which Prefectural Governors are to prepare based on the Basic Policy for Pollution Load Reduction, the followings are to be decided.

(1) Amount of reduction goal for each pollution source in each prefecture (Classified according to industrial, domestic and others).

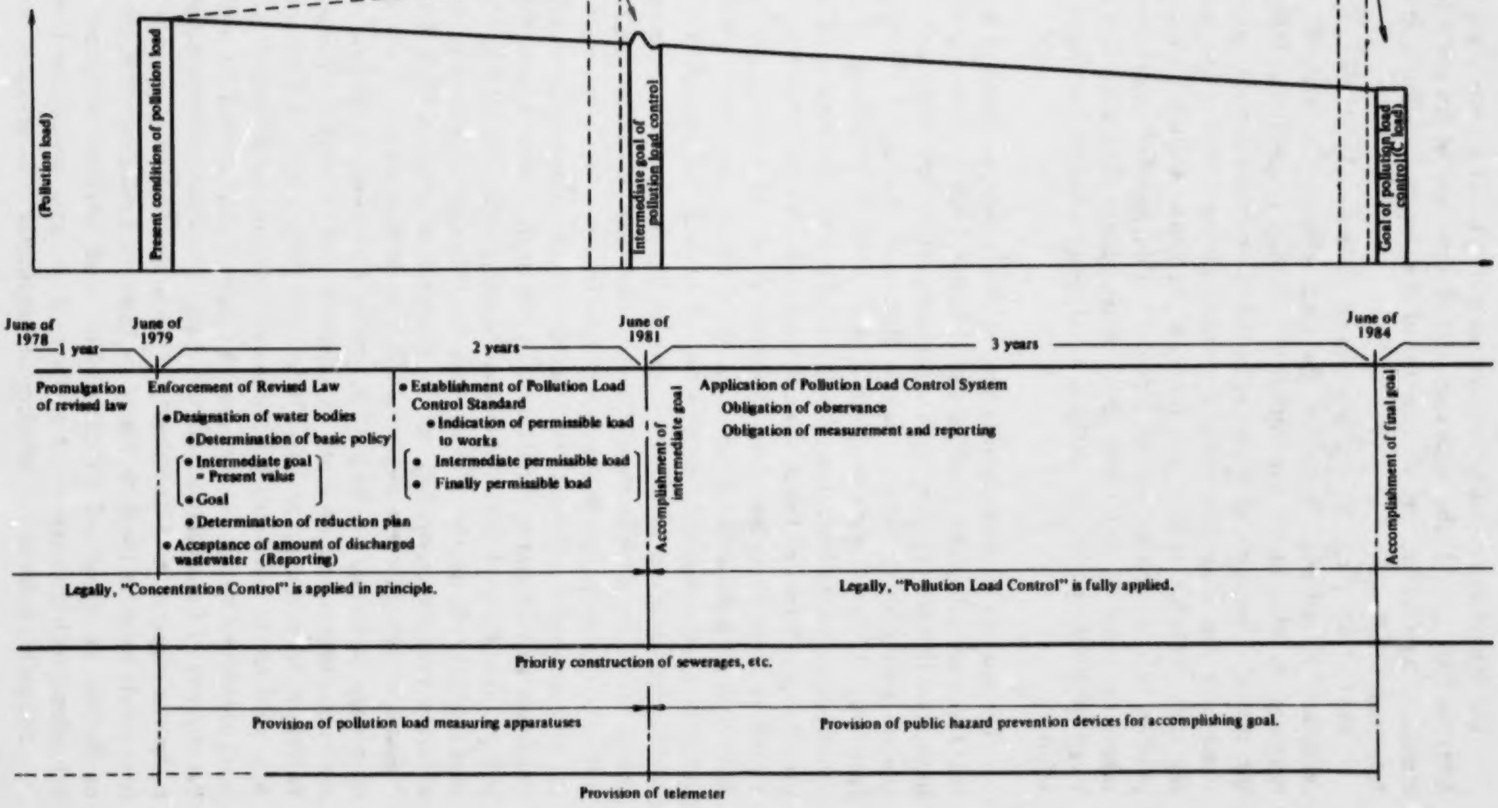
(2) Means of reduction (Pollution Load Control Standard, construction of sewerage and others)

Moreover, in the "Pollution Load Control Standard" which the Prefectural Governor is to decide according to the guideline indicated by the Prime Minister's Office Ordinance and based on the Pollution Load Reduction Plan, the tolerance limits of pollution load per day of discharged wastewater for each works is to be decided and this tolerance limits shall be calculated by multiplying reported amount of discharged wastewater by fixed concentration. (See Fig. 2.2.)

2.5.3 Enforcement of Pollution Load Control and Responsibility of Sewerage

The revised Law on Pollution Load Control is scheduled for enforcement from June of 1979. At that time, various proceedings necessary for the performance of the Pollution Load Control such as the designation of water bodies, determination of the Basic Policy for Pollution Load Control and the Pollution Load Reduction Plan, reporting of the amount of discharged wastewater from specified works within the designated districts, etc., shall be promoted. Based on these factors, the Pollution Load Control Standard for each works will be established in June, 1980 (1 year after the enforcement of the revised Law) and applied to newly established works to begin with. In June of 1981, two years after the enforcement of the revised Law, the Pollution Load Control will be applied extensively to also cover existing works and in 1984 (5 years after the enforcement of the revised Law), the target is to be accomplished.

Fig.-2.3 Imaginary Transfer Diagram Towards "Follution Load Control"


(Pollution load)
Present condition of pollution load
Intermediate goal of pollution load control
Goal of pollution load control (C load)
June of 1978
1 year
June of 1979
2 years
June of 1981
3 years
June of 1984
Promulgation of revised law
Enforcement of Revised Law
• Designation of water bodies
• Determination of basic policy
• Intermediate goal = Present value
• Goal
• Determination of reduction plan
• Acceptance of amount of discharged wastewater (Reporting)
• Establishment of Pollution Load Control Standard
• Indication of permissible load to works
• Intermediate permissible load
• Finally permissible load
Accomplishment of intermediate goal
Application of Pollution Load Control System
Obligation of observance
Obligation of measurement and reporting
Accomplishment of final goal
Legally, "Concentration Control" is applied in principle.
Legally, "Pollution Load Control" is fully applied.
Priority construction of sewerages, etc.
Provision of pollution load measuring apparatuses
Provision of public hazard prevention devices for accomplishing goal.
Provision of telemeter

The Pollution Load Control System will demand on sewerages the responsibility in two aspects; the reduction of pollution load of sewage through its systematical construction and the control of discharged pollution load from sewage treatment plant.

First of all, there lies the problem of sewerage contribution in the aspect of reduction of pollution load. On the establishment of "reduction goal" which becomes the basis in the application of Pollution Load Control System, although the revised Law aims at maintaining the environmental water quality standard concerning the designated items in designated water bodies, the reduction goals for each pollution source and for each prefecture shall be determined on the premise that reductions will be planned within possible enforcement limits, by taking the trend of population and industries in the designated districts, technical levels of sewage treatment, outlook on sewerage construction, etc. into consideration.

The number of prefectures related to Tokyo Bay, Ise Bay and the Seto Inland Sea which are scheduled as objects of Pollution Load Control count 20 and since these areas include the three great urban areas where population and industries are concentrated (Tokyo Area – Tokyo Metropolis, Kanagawa Prefecture, Saitama Prefecture, Chiba Prefecture; Nagoya Area – Aichi Prefecture, Mie Prefecture; Osaka Area – Osaka Prefecture, Kyoto Prefecture, Hyogo Prefecture), about 50% of the total population of Japan concentrate into these areas. Therefore, the pollution generating from these areas are extremely great and particularly since the pollution load from domestic sewage must be reduced by sewerage, it is no exaggeration to say that the systematical construction of sewerage becomes the key of Pollution Load Control. It is due to such circumstances that the prospect of sewerage construction is taken into consideration at the establishment of reduction goal. Therefore, sufficiently investigating the local pollution factors of objective water bodies from the viewpoint of performing the reduction of pollution load most effectively and clarifying the positioning of sewerage as a part of the comprehensive water pollution prevention policy, the reduction goal of which sewerage should take charge must be established based on the 5–Year Plan for Sewerage Construction and individual sewerage projects conformed to the Comprehensive Basin-Wide Sewerage Study Program of the competent area. Particularly, as to the necessity of tertiary treatment of sewage, it should be decided by taking the pollution factors, conditions of sewerage construction, etc. into consideration and by carefully studying its effect. Therefore, in deciding the Basic Policy for Pollution Load Reduction, together with sufficiently investigating beforehand the actual conditions of pollution load from objective districts, the Environment Agency shall consult for adjustment with the Ministry of Construction who is in charge of sewerage administration. Moreover, as for the preparation of Pollution Load Reduction Plan which prefectural Governor is to decide based on the Basic Policy for Pollution Load Reduction, the Department or Bureau in charge of its preparation shall perform sufficient adjustments beforehand with the Department or Bureau in charge of sewerage or river.

Secondly, there is the problem of responsibility of sewerage for controlling pollution load discharged from sewage treatment plant. Since sewerage are handled as specified works similarly as ordinary factories and the like in the Water Pollution Prevention Law, the Pollution Load Control Standard (Permissible limit of discharged pollution load) shall be applied similarly as discharged wastewater from factories and the like for effluent from the sewage treatment plant. Moreover, the Pollution Load Control Standard is to be decided by the Governor based on the guideline of the Environment Agency (Ordinance of Prime Minister's Office).

However, if the water quality of effluent is constant, the discharged pollution load from the sewage treatment plant will enlarge in proportion to the increase of the quantity of effluent, so it is necessary to enable to change the reported quantity of effluent according to the expansion of the treatment facilities. Furthermore, BOD has been mainly used as a pollution index in sewage treatment plant, and COD has been handled as a resembling item of BOD in the Sewerage Law, however, since COD is to be the object in this Pollution Load Control, in the application of the Pollution Load Control Standard for sewage treatment plant, together with sufficiently investigating the actual conditions of removal in the sewage treatment plant into which industrial wastewaters with COD of difficult resolvability, it is also necessary to study the establishment of pre-treatment standard for these COD.

On the other hand, for furthering the effect of Pollution Load Control on the sewerage side, it is necessary to exert efforts so that the increase of pollution load generated from the treatment area of sewerage may be controlled to the utmost. For this purpose, efforts such as providing an incentive on pollution load reduction to sewerage users by adopting a progressive user charges system for large quantity dischargers, and by adding a water quality surcharge for wastewater of high concentration, performing enlightenment for pollution load reduction and others become necessary.

3. CONCLUSION

We have hitherto related on recent trends and the institutional structures of water pollution control in Japan, and particularly with the introduction of the Pollution Load Control System, systematical construction of sewerage in designated districts is to be obligated and the role of sewerage as a water pollution prevention policy has become increasingly greater.

As a consequence, the necessity of co-operation between Environment Agency and Ministry of Construction in national level and between Department or Bureau of Environment and Sewerage in prefectural and municipal levels in promoting water pollution control has become much more stronger than ever.

CHAPTER 3
FINANCING OF SEWAGE WORKS – USER CHARGES

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1. INTRODUCTION

Financing of sewage works has gradually changed corresponding with the variety of the function of sewerage.

The principle of financing sewage works had been studied three times by the Study Committee for Sewage Works Financing organized at Japan City Center in 1960, 1966 and 1973. The recommendations for adequate financing of sewage works for each period were made by the committee. The recommendation by the 1st committee in 1960 was made from the viewpoint that sewerage was a basic facility for city and brought about improvement of city environment and public health by its function of storm water drainage, wastewater drainage and purification. The 2nd committee in 1966 stressed that the function of sewerage had been varied and complicated, especially the sewerage provision in large cities had urgently necessitated for water pollution control. Furthermore, the 3rd committee in 1973 made the recommendation from the viewpoint that reflecting serious water pollution problems the social demand for sewerage for the purpose of water quality preservation of public water bodies and effective use of water resources had more and more swollen.

Based on these recommendations, the five-year-plans for sewerage construction and other policies were enforced.

After the recommendation by the 3rd committee in 1973, however, the circumstances of sewerage administration have remarkablly changed; for instance, (1) the pollution load control system has come to introduce in order to improve water quality in widely closed water bodies, (2) the maintenance and operation of sewerage are becoming an important problem, (3) through the cognizance of limitation of water resources, the social request for effective use of water including reuse of wastewater has enhanced.

Under the circumstances, the 4th Study Committee for Sewage Works Financing was organized in June this year and is continuing to deliberate the financing of sewage works for new era, including the policies for systematic and effective construction of sewerage facilities and the principle of managing sewerage systems.

This paper describes the outline of the status quo of sewage works financing in Japan, and introduces the actual condition and the principle of the user charges which is closely related to the management of sewerage systems.

2. OUTLINE OF SEWAGE WORKS FINANCING AND ITS TENDENCY

2.1 FINANCIAL SOURCES FOR CONSTRUCTION COST

2.1.1 Composition of Financial Sources for Construction Cost

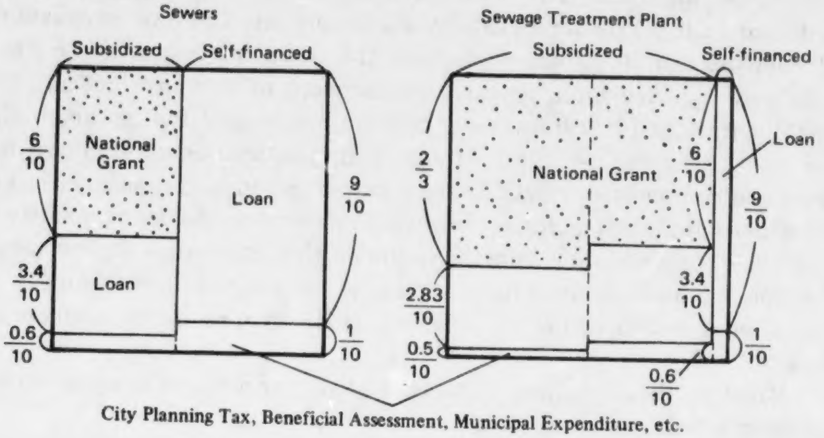
Sewerage is classified into three types; public sewerage, regional sewerage and storm water channel, but in this paper the former two are the object for description.

Public sewerage is managed by municipality, while regional sewerage which is a trunk sewerage covering more than two municipalities is managed by prefectural government.

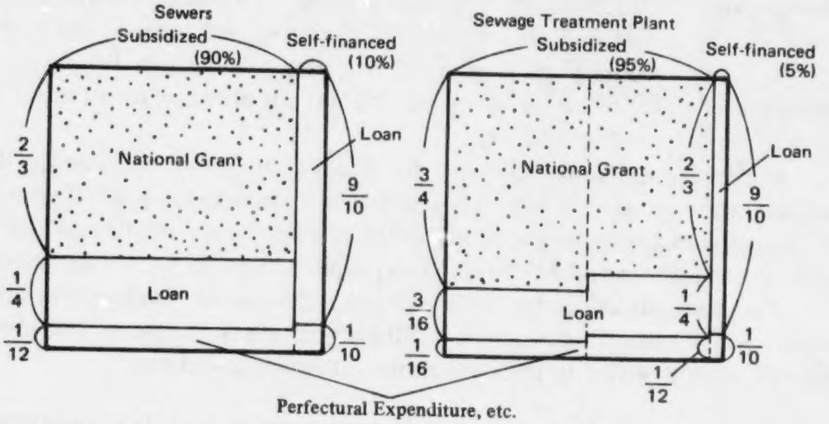
The financial sources for construction cost of sewerage facilities consist of national grant, loan, general tax, city planning tax, prefectural grant, beneficial assessment, etc. The composition of these financial sources for public sewerage and regional sewerage are shown in Figure 2.1.

Fig. -2.1 Composition of Financial Sources for Construction Cost in Sewage Works

(1) Public Sewerage



(2) Regional Sewerage



a. National Grant

Since sewerage facilities should be provided as a part of water pollution control measures from the national point of view, the rate of national grant for sewerage construction cost has gradually been raised until now. The rate of national grant is 6/10 (2/3 for sewage treatment facilities) for public sewerage, and 2/3 (3/4 for sewage treatment facilities) for regional sewerage since 1974.

On the other hand, the objects of national grant are confined to main sewer, pumping station and sewage treatment plant, and are fixed by the rule of Ministry of Construction. According to this rule, the rate of the construction cost of the facilities to be subsidized by the national government to the total construction cost is 60% (45% for designated cities, 75% for general cities) for public sewerage in the 4th Five Year Plan for Sewerage Construction.

b. Loan

Loan is permitted for the certain rates of construction cost and compose of a main financial source for construction cost. Appropriation rates of loan for sewage works are shown in Table 2.1.

Table -2.1 Appropriation Rate of Loan

Classification			National Grant	Local Burden	Loan out of Local Burden
Public Sewerage	Sewers	Subsidized	6/10	4/10	4/10 x 8.5/10 = 3.4/10
		Self-financed	–	10/10	10/10 x 9/10 = 9/10
	Sewage Treatment Plant	Subsidized	2/3	1/3	1/3 x 8.5/10 = 2.83/10
		Self-financed	–	10/10	10/10 x 9/10=9/10
Regional Sewerage	Sewers	Subsidized	2/3	1/3	1/3 x 3/4 = 1/4
		Self-financed	–	10/10	10/10 x 9/10 = 9/10
	Sewage Treatment Plant	Subsidized	3/4	1/4	1/4 x 3/4 =3/16
		Self-financed	–	10/10	10/10 x 9/10 = 9/10

The funds for sewage works loan are classified into three categories; governmental funds, finance corporation funds and private funds (market collection funds, bank connection funds). The shares of each category in 1978 are 30% in governmental, 23% in corporation and 47% in private, respectively. This shows that the governmental funds of low interest tend to decrease while the private funds of high interest are inclined to increase.

c. Beneficial assessment

The beneficial assessment for sewage works is levied on the owners of land which is located within the drainage area of a public sewerage, based on a municipal ordinance in accordance with the City Planning Law.

The number of municipalities that adopt this financial system are 295 out of 547 municipalities which had been approved their projects of public sewerage systems at the end of 1976, and the unit amount of beneficial assessment is mostly 100 to 300 yen per square meter of assessible land. The total amount of benefisial assessment in 1976 throughout the country occupied about 1.1% of the total construction cost, and about 2.5% of the construction cost in the municipalities in which beneficial assessment had been collected.

d. City planning tax

City planning tax can be levied by a municipality on the owners of land and buildings which are located within a city planning area in order to appropriate to the cost of city planning projects. Therefore, city planning tax can be appropriated to other city planning projects than public sewerage. The city planning tax appropriated to the public sewerage in 1976 occuped about 1.3% of the total construction cost throughout the country, that is approximately same percentage as the beneficial assessment.

e. Prefectural grant

The number of prefectural governments that deliver grant to municipalities that execute public sewerage works is 12 in 1976 (Total number of prefectural government is 47.)

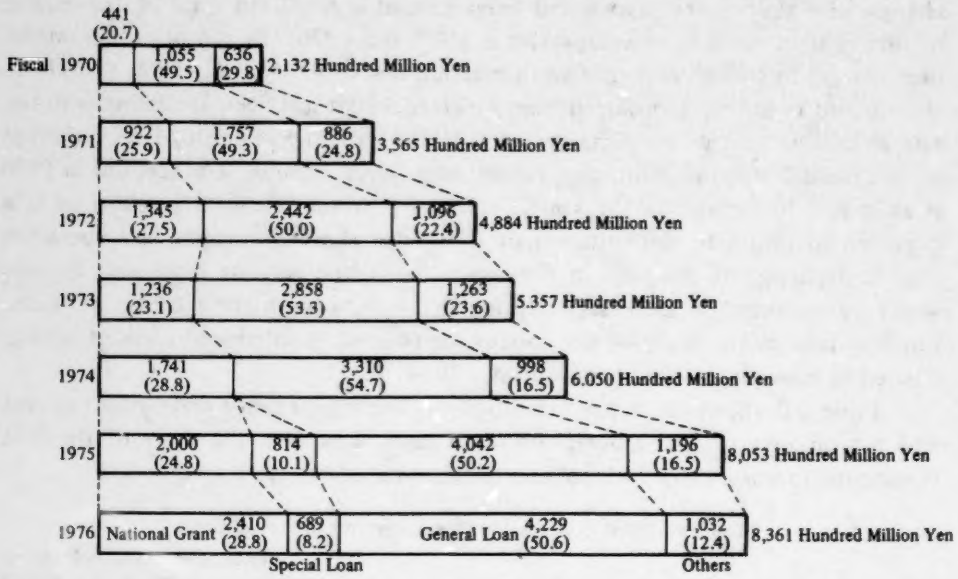
The percentage of the prefectural grant in the total construction cost of public sewerage in 1976 was about 0.5% and have tended to decrease since 1974.

Construction cost of a regional sewerage after subtracting national grant is as a general rule borne fifty-fifty by a prefectural government and related municipalities. Allotting method among related municipalities is generally by the ratio of planned volume of wastewater or planned drainage area of each municipality.

2.1.2 Tendency of Financial Sources for Construction Cost

The shares of financial sources for sewerage construction cost for each fiscal year from 1970 to 1976 are shown in Fig. 2.2. Reflecting the public character and the urgency as water pollution control measures of sewage works, the amount of construction cost has been increasing year by year. It is noted that since 1974 the share of local government expenses other than the floation of loan has been remarkably lightened. This had been caused by the improvement of the rate of national grant and the appropriation rate of loan in 1974, and furhtermore, the introduction of special loan system in 1975. The special loan system is a financial system applied to the construction cost of sewage treatment plants of public sewerage concerning to the public water bodies in which environmental water quality standards have been established. In order to promote sewage works to attain environmental water quality standards while national grant to evenly deliver, the national grant is advanced by the special loan which is paid in installment by national grant for five years.

Fig. -2.2 Actual Conditions of Financial Sources for Construction Cost in Sewage Works



2.2 FINANCIAL SOURCES FOR MAINTENANCE AND OPERATION COST

2.2.1 Actual Conditions of Financial Sources for Maintenance and Operation Cost

The actual conditions of financial sources for maintenance and operation cost of public sewerage throughout the country are shown in Table 2.2, which indicates the the maintenance and operation cost has been remarkably increasing year by year. It is pointed out that the reasons for this are the increase of the administrative expenditure such as the cost for surveillance and guidance for pretreatment facilities, the expense for propagation of flush toilet, and the requirement of high quality treatment with strengthening of water quality control, in addition to the increase of sewerage facilities.

Table -2.2 Actual Conditions of Financial Sources for Maintenance and Operation Cost

(Unit: Million Yen)

Fiscal Year	Maintenance and Operation Cost (A)	Increasing Rate	Financial Sources for Maintenance and Operation Cost						(B)/(A)	(C)/(A)
			User Charges (B)	Increasing Rate	Money from General Account (C)	Increasing Rate	Others		
1972	61,578	100	31,246	100	24,850	100	5,482	50.7	40.4
1973	83,244	135	37,211	119	32,038	129	13,995	44.7	38.5
1974	123,007	200	41,532	133	53,233	214	28,242	33.8	43.3
1975	134,585	219	65,354	209	60,502	243	8,729	48.6	45
1976	257,232	255	96,703	309	–	–	–	61.5	–

The most part of the maintenance and operation cost is covered by user charges and the money transferred from general account. In spite of the raising of user charges in many municipalities in 1975 and 1976, the appropriation rate of user charges to the maintenance and operation cost in 1976 is only 61%. Therefore, the amount of money transferred from general account had been increasing with the rate exceeding that of user charges until 1975. With the revision of user charges in 1975 and 1976, the amount of money transferred from general account in 1976 is estimated to remain on the same level as a previous year and the share of it is expected to return to the situation of 1973. But the maintenance and operation cost is presumed to increase in the future, while the revenue from user charges won't be expected to increase excepting the revenue with the increase of users. For this reason, the share of the money transferred from general account is anticipated to increase like the condition since 1974.

Table 2.3 shows the actual conditions of the maintenance and operation cost and the amount of loan redemption. It is noted here that the share of the cost concerning to wastewater drainage and purification comes up to 43% to 63%.

Table 2-3 Maintenance and Operation Cost in Main Cities

(Fiscal 1976, Unit: Million Yen)

Classification / Name of city	Maintenance and Operation Cost for Wastewater A	A/D %	Loan Redemption for Wastewater B	B/D %	Loan Interest for Wastewater C	C/D %	Total for Wastewater D	Cost for Wastewater / Total Cost %	Loan Balance at the End of Fiscal 1976
Sapporo	3,046.6 (2,193.6)	29.3 (48.2)	3,822.4 (1,223.2)	36.8 (26.9)	3,531.1 (1,130.0)	34.0 (24.9)	10,400.3 (4,546.8)	43.7	55,757.3
Sendai	1,164.8 (815.1)	36.3 (72.2)	227.5 (68.3)	7.1 (6.0)	819.1 (245.7)	25.5 (21.8)	2,211.5 (1,129.1)	51.1	14,648.4
Kawasaki	2,420.0 (1,621.4)	49.8 (52.9)	598.0 (35.8)	12.3 (11.6)	1,837.8 (1,102.7)	37.8 (35.8)	4,855.8 (3,082.9)	63.5	33,918.2
Nagoya	9,610.0 (6,823.1)	48.3 (59.1)	2,441.6 (1,123.1)	12.3 (9.7)	7,839.6 (3,606.2)	39.4 (31.2)	19,891.2 (11,552.4)	58.1	115,494.2
Kyoto	4,757.0 (3,994.2)	43.4 (60.5)	943.4 (396.2)	8.6 (6.0)	5,266.7 (2,212.0)	(48.0) (33.5)	10,965.1 (6,602.4)	60.2	89,494.8
Osaka	16,424.0 (10,628.9)	55.4 (64.7)	2,643.9 (1,208.3)	8.9 (7.4)	10,604.7 (4,591.8)	35.7 (27.9)	29,676.8 (16,429.0)	55.9	157,912.8
Okayama	656.0 (459.2)	50.4 (70.3)	109.0 (32.7)	8.4 (5.0)	(536.4) (160.9)	41.2 (16.1)	1,301.4 (652.8)	50.2	8,985.4
Hiroshima	2,062.6 (1,443.8)	64.0 (80.6)	197.8 (59.3)	6.1 (3.3)	964.2 (289.3)	29.9 (16.1)	3,224.8 (1,792.4)	55.6	14,058.2
Kitakyushu	1,332.8 (1,095.6)	22.8 (36.8)	980.2 (409.7)	16.8 (13.8)	3,527.1 (1,474.3)	60.4 (49.5)	5,840.2 (2,979.6)	51.0	48,805.8

2.2.2 Actual Conditions of the Amount of Money Transferred from General Account

The money transferred from general account to sewage works account is appropriated not only to maintenance and operation cost but also construction cost and loan redemption. Table 2.4 shows the actual conditions of the amount of money transferred from general account for each population rank of municipalities.

Table -2.4 Amount of Money from General Account

(Unit: Hundred Million Yen)

Population Rank of Municipalities	Fiscal Year	Amount of Money from General Account					Total Cost for Maintenance and Operation	Remarks
		for Construction Cost	for Loan Redemption	Sub-Total	for Maintenance and Operation Cost	Total		
More than a Millon	1972	(100) 320	(100) 354	(100) 674	(100) 119	(100) 793		
	1973	(93) 296	(126) 446	(110) 742	(122) 145	(100) 887		
	1974	(57) 183	(174) 616	(119) 799	(211) 251	(132) 1050		
	1975	(80) 256	(225) 795	(156) 1051	(224) 267	(166) 1318		
100,000 to 1,000,000	1972	(100) 252	(100) 159	(100) 411	(100) 112	(100) 523		
	1973	(97) 245	(135) 215	(112) 460	(135) 151	(117) 611		
	1974	(83) 208	(167) 265	(115) 473	(215) 241	(137) 714		
	1975	(71) 178	(214) 340	(126) 518	(267) 299	(156) 817		
Less than 100,000	1972	(100) 162	(100) 24	(100) 186	(100) 19	(100) 205		
	1973	(60) 98	(163) 39	(74) 137	(126) 24	(79) 161		
	1974	(63) 102	(204) 49	(81) 151	(205) 39	(93) 190		
	1975	(62) 101	(288) 69	(91) 170	(205) 39	(102) 209		
Total	1972	(100) 734	(100) 538	(100) 1272	(100) 249	(100) 1521	(100) 616	
	1973	(87) 639	(130) 700	(105) 1339	(129) 320	(109) 1659	(135) 832	
	1974	(67) 493	(173) 930	(112) 1423	(214) 532	(129) 1955	(200) 1230	
	1975	(73) 535	(224) 1205	(137) 1740	(243) 605	(154) 2345	(219) 1346	

(Note) Figures in the brackets show the increasing rates of each year in case of 100 for fiscal 1972.

This table indicates that the appropriation to the construction cost has gradually been decreasing, while the appropriation to the loan redemption has remarkably been increasing, particularly in municipalities of small population rank. Loan redemption consists of principal and interest, and the share of the interest is very high as shown in before-mentioned Table 2.3. With the progress of sewerage construction works hereafter, loan redemption will become a main oppressing factor for financing of sewage works. As mentioned before in 2.1.1, b, the tendency that the share of governmental funds has been lowering while the share of private funds has been rising in sewage works loan will further burden general account.

3. SEWERAGE USER CHARGES

3.1 EXISTING SYSTEMS OF USER CHARGES

3.1.1 Type of User Charges System

Although user charges systems in Japan are various reflecting a special character of each municipality, these systems are classified into two types.

a. Water charges proportional system

Water charges proportional system is the system in which sewerage user charges are calculated as a certain percentage of water charges. But in this case a separate system is usually adopted to the wastewater from other supply of water such as wellwater.

In the municipalities adopting this system, the identified charges system for both water supply and sewerage is familiar to citizen. In addition to this, water charges proportional system has considerable merits of collecting convenience and easy computation of the charges. However, since this system has the defect that the cost price is not reflected to the user charges, municipalities adopting this system is decreasing in number (See Table 3.1).

Table 3-1 Classification of User Charges System [1] (Sampled for 85 Cities)

Classification		End of Fiscal 1971	End of Fiscal 1972	End of Fiscal 1973	End of Fiscal 1974	End of Fiscal 1975	June of 1976
Wastewater from Water Supply	(A) Water Charges Proportional System	22	20	21	18	17	15
	(B) Wastewater Quantity Conformable System	55	59	62	66	67	69
	Other System	3	3	2	1	1	1
	Total	80	82	85	85	85	85
Wastewater from Well water and Others	(A) Separate System from One for Wastewater from Water Supply	17	17	17	16	15	14
	(B) Same System as One for Wastewater from Water Supply	45	45	51	55	58	61
	Other System	16	18	16	14	12	10
	Total	78	80	84	85	85	85
Average Rate of Sewerage User Charges to Water Charges Among the Cities Where Water Charges Proportional System Are Applied (%)		40.2	42.0	40.0	37.4	37.2	39.2

b. Wastewater quantity conformable system

Wastewater quantity conformable system is the system in which sewerage user charges are calculated in conformity with the quantity of discharged wastewater. This system has the merits that the cost price corresponding with the quantity of discharged wastewater can be reflected to the user charges, and furthermore, by adopting progressive charges system, the promotion of reducing wastewater from large quantity dischargers can be expected. The quantity of discharged wastewater for this charges system is recognized by inspecting a water consumption meter in most cities (See Table 3.1).

3.1.2 Progressive Charges System

Among the municipalities adopting water charges proportional system or wastewater quantity comformable system, the progressive charges system in which user charges per unit quantity of discharged wastewater goes up as the quantity increases, the uniform charges system in which user charges per unit quantity is constant, and the diminished charges system in which user charges per unit quantity becomes cheaper for larger quantity dischargers, are adopted. However, the municipalities adopting the progressive charges system are rapidly increasing in number (See Table 3.2 and Table 3.3).

Table -3.2 Classification of User Charges System [II]
(In case of (A) and (B) for Wastewater from Water Supply in Table 3-1)

Classification		End of Fiscal 1971	End of Fiscal 1972	End of Fiscal 1973	End of Fiscal 1974	End of Fiscal 1975	June of 1976
Without Basic Charges	Uniform Charges System	16	18	20	20	17	8
	Progressive Charges System	3	4	5	5	9	16
	Diminished Charges System	6	5	5	5	5	1
	Both Progressive and Diminished Charges System	1	1	1	1		
	Sub-Total	26	28	31	31	31	25
With Basic Charges	Uniform Charges System	40	37	37	32	25	13
	Progressive Charges System	8	8	13	19	25	42
	Diminished Charges System	3	3	2	2	3	4
	Sub-Total	51	51	52	53	53	59
Total		77	79	83	84	84	84
Average Number of Progressive Steps	Without Basic Charges	2.3	2.5	2.8	6.4	5.9	6.1
	With Basic Charges	2.8	2.8	2.8	3.4	4.2	4.8

Table -3.3 Actual Conditions of Application of Progressive Charges System and Water Quality Surchages System
(End of Fiscal 1976)

Number of Municipalities Applying Progressive Charges System	
End of Fiscal 1972	17
End of Fiscal 1974	40
End of Fiscal 1976	126

Number of Municipalities Applying Water Quality Surcharges System	
End of Fiscal 1972	9
End of Fiscal 1974	18
End of Fiscal 1976	31

3.1.3 Water Quality Surcharges System

Water quality surcharges are the additional user charges which are imposed to dischargers who dischage wastewater of higher concentration than a certain level. The merits of adopting water quality surcharges system are that the cost recovery can be made by fair imposition of user charges in accordance with the conditions of discharged wastewater, and that the effect of encouraging efforts to improve the quality of wastewater of high concentration to the dischargers can be expected. Therefore, the municipalities adopting water quality surcharges system are steadily increasing in number (See Table 3.3)

3.1.4 Level of User Charges

Since user charges systems are various among municipalities, it is difficult to simply compare the level of user charges in one municipality with others. Table 3.4 shows the distribution of user charges in ordinary household which discharges 20 m^3 per month. The table indicates that user charges in average household distribute in considerably wide range centering from 400 yen to 599 yen throughout the country and user charges in large cities are generally cheaper.

Table -3.4 Distribution of User Charges in Ordinary Household (End of Fiscal 1976)

User Charges for 20 m³ per Month in Ordinary Household (Yen)	~399	400~599	600~799	800~999	1000~	Total
Number of Municipalities (Number of Specified Cities)	88 (8)	93 (2)	47 (0)	11 (0)	10 (0)	249 (10)

3.1.5 Sewage Works Expenditures to be covered by User Charges

The expenditures to be covered by user charges are a depreciation cost and a loan interest besides a maintenance and operation cost. The actual conditions of the expenditures included in the estimation of user charges in the municipalities which had collected user charges at the end of fiscal 1976 are shown in Table 3.5. The table indicates that the number of municipalities which include the depreciation cost and the loan interest besides the maintenance and operation cost in the estimation of user charges are 103 out of 249, and that 28 municipalities include all of these cost in the estimation of the user charges only for the part of wastewater exceeding a certain quantity (specified wastewater).

Table -3.5 Actual Conditions of Expenditures Covered by User Charges (End of Fiscal 1976)

Expenditures Covered by User Charges, Except for Maintenance and Operation Cost	Number of Municipalities Adopting to Both Ordinary and Specified Wastewater	Number of Municipalities Adopting only to Specified Wastewater	Total
Depreciation Cost and Loan Interest	58	26	84
Depreciation Cost Only	9	1	10
Loan Interest Only	8	1	9
Total	75	28	103

In this case, the standard quantity of specified wastewater is mostly over 1,000 m³ or 500 m³ per month. The percentage of the quantity of specified wastewater to the total is about 20%, while in most cases the user charges from specified wastewater is over 40% of the total revenue of user charges, reflecting the adoption of the progressive charges system and the water quality surcharges system (See Table 3.6)

Table -3.6 Amount of Specified Wastewater and Revenue of User Charges from Specified Wastewater (Fiscal 1976)

Name of City	Sapporo	Yokohama	Kobe	Sendai
Definition of Specified Wastewater (More than Cubic Meter per Month)	1,000	501	500	1,000
a. Total Amount of Specified Wastewater (Thousand Cubic Meter)	31,885	37,160	20,489	9,158
b. Rate of Amount of Specified Wastewater to Total (%)	32.9	23.6	18.1	19.9
c. Revenue of User Charges from Specified Wastewater (Thousand Yen)	1,899,809	1,518,426	1,521,805	192,875
d. Rate of Revenue from Specified Wastewater to Total (%)	52.0	42.7	47.2	26.7
c/a	60	41	74	21
d/b	1.6	1.8	2.6	1.1
Progressive Charges System	○	○	○	○
Water Quality Surcharges System		○	○	

3.2 WHAT USER CHARGES SHOULD BE

3.2.1 Basic Principle of User Charges

Since the users of sewerage facilities can be distinguished from others, it is necessary from the viewpoint of effective management of the sewerage facilities and fair charge of the expenditures to levy user charges in accordance with the conditions of the use. On the other hand, it is necessary to burden needed cost to dischargers of wastewater accompanied with enterprising activities in conformity with 'the pollutor-pay-principle' (PPP). However, since it is difficult to determine dischargers and their quantities and qualities of wastewater at the beginning of sewerage construction, it is adequate to burden needed cost through user charges.

Article 20 of Sewerage Law stipulates that the superintendent of the public sewerage is able to collect user charges prescribed by the municipal ordinance from its users in conformity with the following principles.

1) To be appropriate in accordance with the conditions of the use such as the quantity and the quality of wastewater.
2) Not to exceed the reasonable cost price under the efficient management.
3) To be definitely determined with the fixed rate or the fixed amount.
4) Not to unjustly discriminate specific users.

From the above-mentioned point of view, the followings should be the basic principle of sewerage user charges.

a. Range of expenditures to be covered by user charges

Since the appraisal of the conditions of the use should be made with respect to the wastewater which is discharged to the sewerage by users, and the construction cost of sewerage should be paid with public expenditure excepting the beneficial assessment and the burden to be conformed to PPP, the range of expenditures to be covered by user charges should be the maintenance and operation cost as for the ordinary wastewater (the whole of domestic sewage and the part of wastewater under a certain quantity from industrial and business activities).

As for the specified wastewater (the part of wastewater over a certain quantity from industrial and business activities), the repayment cost of the facilities and the loan interest in addition to the maintenance and operation cost should be included in the expenditures to be covered by user charges.

b. Adoption of water quality surcharges system

In order to aim the thoroughness of PPP and the fairness of the burden among users, water quality surcharges should be added to water quantity charges.

In this case, the items of water quality and the range of concentration to be considered in water quality surcharges system should be limited within the bounds not to extremely obstruct sewerage functions or not to impair sewerage facilities and to be able to keep the water quality standards for effluent from sewage treatment plant. The items and the concentration exceeding these limitations and toxic pollutants should be removed by pretreatment facilities prior to be discharged into a public sewerage.

c. Adoption of progressive charges system

It is desirable to adopt the progressive charges system in which the unit charges go up progressively as the quantity of discharged wastewater increases, within the bounds that the total amount of user charges does not exceed the cost price.

The progressive charges system together with the water quality surcharges system acts as the incentive to the efforts for reducing the quantity and for improving the quality of the wastewater discharged to sewerage, and it is expected to be useful for saving and effective use of water resources, recovery of resources and effective management of sewerage facilities.

d. User charges for tertiary treatment

Since the effluent from tertiary treatment should be in principle returned to a public water body as a part of water management, the cost for tertiary treatment should be covered by public expenses and be excluded from the expenditures to be covered by user charges, excepting the cost for the specified wastewater.

As for the specified wastewater, the cost for tertiary treatment (including repayment cost of facilities and loan interest besides maintenance and operation cost) should be covered by user charges, in conformity with PPP.

3.2.2 Bases for Calculation of User Charges

a. User charges for ordinary wastewater

(1) Water Quantity Charges

$$\text{Average Charges per m}^3 = \frac{\text{Annual Cost for Maintenance and Operation}}{\text{Annual Quantity of Ordinary Wastewater (m}^3\text{)}}$$

(2) Water Quality Surcharges

None

b. User charges for specified wastewater

(1) Water Quantity Charges (including the user charges for tertiary treatment for BOD (or COD) and SS)

Average Charges per m³

$$= \frac{\dfrac{\begin{array}{c}\text{Part of Construction Cost and Loan}\\ \text{Interest for Specified Wastewater}\\ \text{to be Covered by Water Quantity Charges}\end{array}}{\text{Duration Years}} + \begin{array}{l}\text{Part of Maintenance}\\ \text{and Operation Cost for}\\ \text{Specified Wastewater}\\ \text{to be Covered by Water}\\ \text{Quantity Charges}\end{array}}{\text{Annual Quantity of Specified Wastewater (m}^3\text{)}}$$

(2) Water Quality Surcharges

Average Charges per m^3

$$= \frac{\dfrac{\text{Part of Construction Cost and Loan Interest for Specified Wastewater to be Covered by Water Quality Surcharges}}{\text{Duration Years}} + \text{Part of Maintenance and Operation Cost for Specified Wastewater to be Covered by Water Quality Surcharges}}{\text{Annual Quantity of Specified Wastewater to be Subjected to Water Quality Surcharges } (m^3)}$$

(Note) As to the user charges for tertiary treatment for other items of water quality than BOD (or COD) and SS, the user charges should be calculated based on the same idea of the above-mentioned formula, where the water quality exceeds that of domestic sewage.

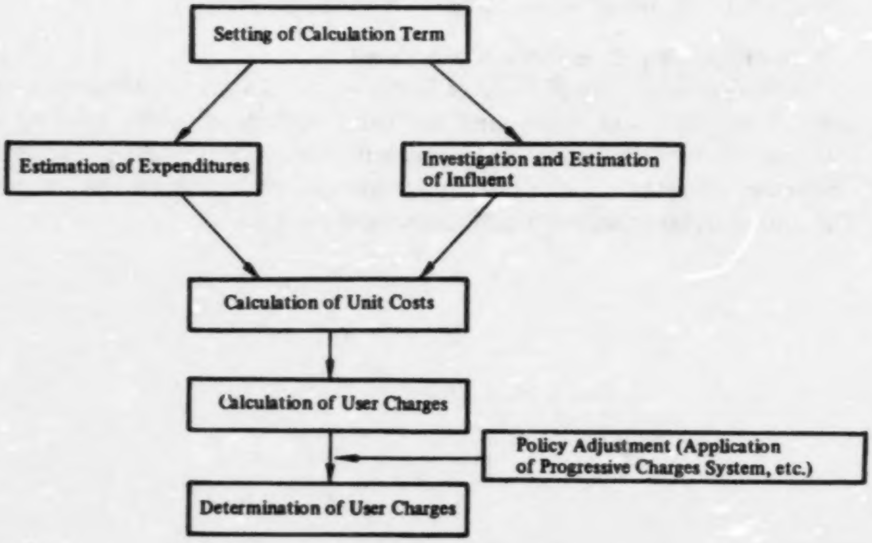
Calculation term of user charges is usually set up in the range of three to five years. At the determination of the calculation term, it is necessary to consider the equalization of the burden of the initial cost over the future, while the estimation of the cost price during the calculation term has to be possible.

Where a preceding investment is quite huge, a part of the initial cost which cannot be covered by the present user charges is advanced by public expenditure and is to be recovered by the user charges in later years.

3.2.3 Method of Calculation of User Charges

The process of determining user charges in accordance with the above-mentioned bases of calculation is shown in Figure 3.1.

Fig. -3.1 Process of Determining User Charges

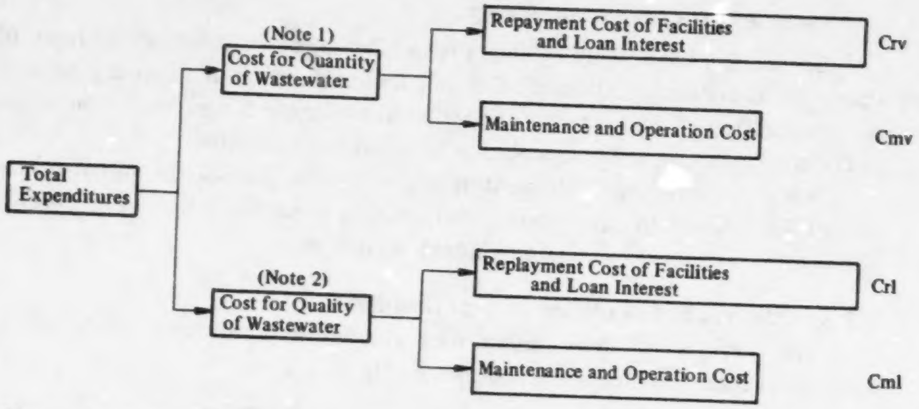

Setting of Calculation Term
Estimation of Expenditures
Investigation and Estimation of Influent
Calculation of Unit Costs
Calculation of User Charges
Policy Adjustment (Application of Progressive Charges System, etc.)
Determination of User Charges

a. Estimation of expenditure

The capital cost (depreciation cost and loan interest) and the maintenance and operation cost of the sewerage facilities are estimated extending over the calculation term, which should be determined by considering the certainty for estimation of the cost price and the income and outgo program as well as the administrative stability, and the reasonable term is about three to five years. In this case, it is necessary for estimating the stability of user charges to calculate user charges for several cases of the term.

The total expenditures are classified into some elements indicated in the bases for calculation of user charges, as shown in Fig. 3.2.

Fig. -3.2 Classification of Expenditures


Total Expenditures
(Note 1)
Cost for Quantity of Wastewater
Repayment Cost of Facilities and Loan Interest
Crv
Maintenance and Operation Cost
Cmv
(Note 2)
Cost for Quality of Wastewater
Replayment Cost of Facilities and Loan Interest
Crl
Maintenance and Operation Cost
Cml

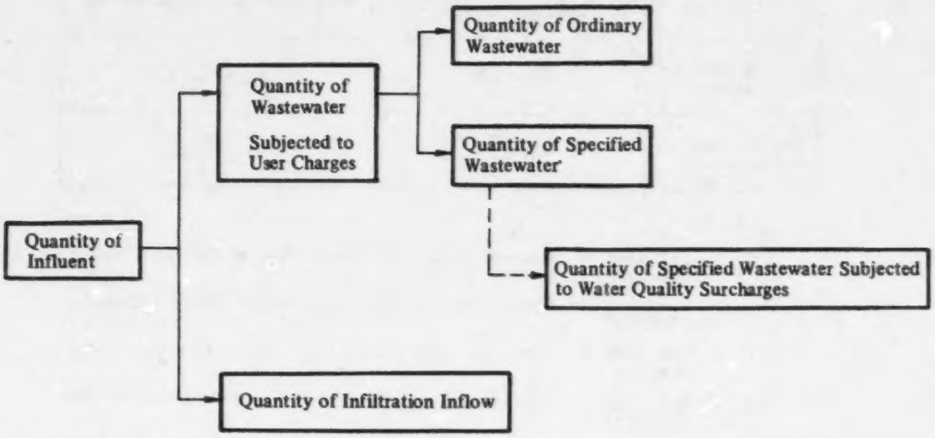
(Note 1) Cost for Sewers, Pumping Stations and Primary Treatment Facilities
(Note 2) Cost for Secondary Treatment Facilities and Sludge Handling Facilities

b. Investigation and estimation of influent

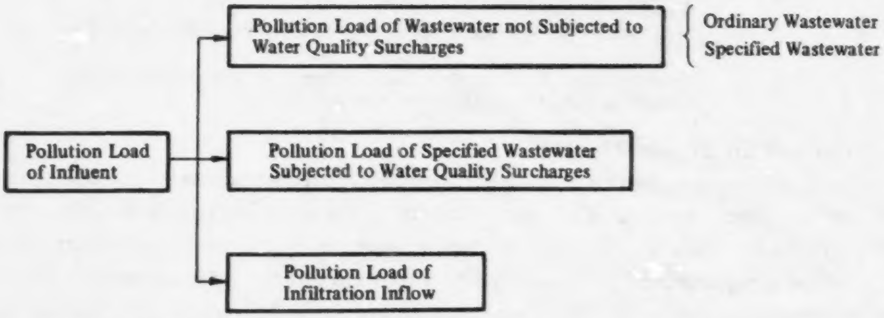
Sewerage user charges are classified into the charges for ordinary wastewater and for specified wastewater, and the latter consists of water quantity charges and water quality surcharges. Therefore, both the quantity and the quality of each wastewater flowing into a sewage treatment plant should be investigated. The process of investigation of influent is shown in Fig. 3.3.

Fig. 3.3 Investigation and Estimation of Influent

(Investigation of Quantity)


Quantity of Influent
Quantity of Wastewater Subjected to User Charges
Quantity of Ordinary Wastewater
Quantity of Specified Wastewater
Quantity of Specified Wastewater Subjected to Water Quality Surcharges
Quantity of Infiltration Inflow

(Investigation of Quality)


Pollution Load of Influent
Pollution Load of Wastewater not Subjected to Water Quality Surcharges
Ordinary Wastewater
Specified Wastewater
Pollution Load of Specified Wastewater Subjected to Water Quality Surcharges
Pollution Load of Infiltration Inflow

c. Calculation of unit costs

Unit costs mean the cost per cubic meter of wastewater and the cost per gram of pollution load, and can be calculated by dividing the expenditures estimated at a. by the quantities and the pollution loads investigated at b. The calculation formulas of the unit cost for each expenditure are shown in Table 3.7.

Table -3.7 Calculation of Unit Costs

Item of Cost	Repayment Cost of Facilities and Loan Interest	Maintenance and Operation Cost
Cost for Quantity of Wastewater	$U_{rv} = \frac{C_{rv}}{V}$	$U_{mv} = \frac{C_{mv}}{V}$
Cost for Quality of Wastewater	$U_{rl} = \frac{C_{rl}}{L}$	$U_{ml} = \frac{C_{ml}}{L}$

(Notes)

U_{rv} : Unit Cost for Repayment and Loan Interest out of Cost for Quantity (Yen/m^3)

U_{mv} : Unit Cost for Maintenance and Operation out of Cost for Quantity (Yen/m^3)

U_{rl} : Unit Cost for Repayment and Loan Interest out of Cost for Quality (Yen/gr.)

U_{ml} : Unit Cost for Maintenance and Operation out of Cost for Quality (Yen/gr.)

C_{rv} : Cost for Repayment and Loan Interest out of Total Cost for Quantity during Calculation Term

C_{mv} : Cost for Maintenance and Operation out of Total Cost for Quantity during Calculation Term

Crl : Cost for Repayment and Loan Interest out of Total Cost for Quality during Calculation Term

Cml : Cost for Maintenance and Operation out of Total Cost for Quality during Calculation Term

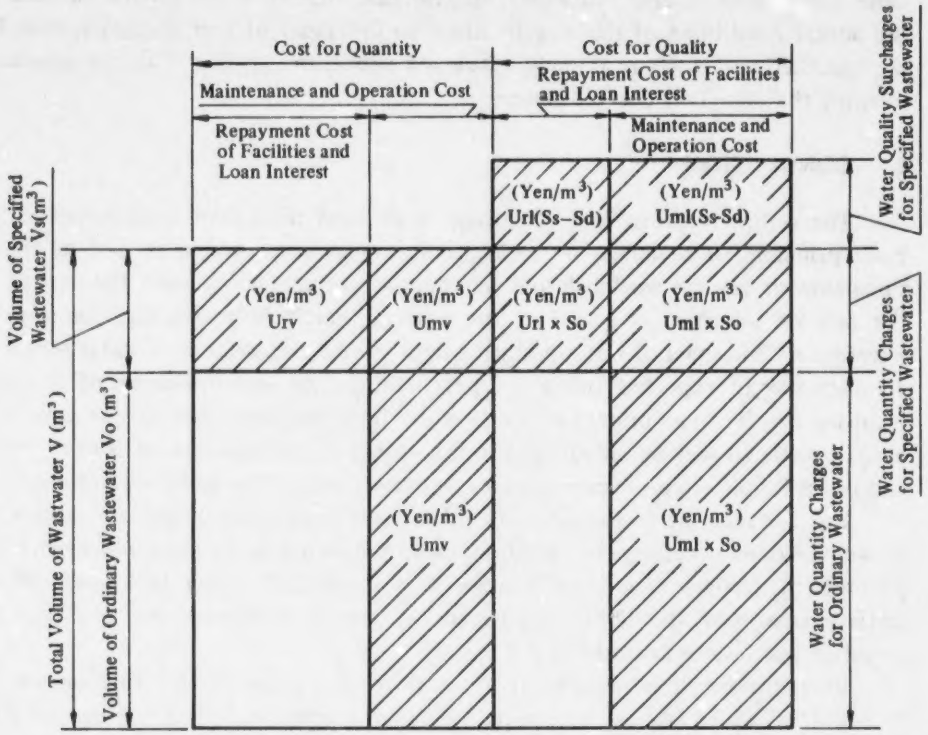
V : Total Volume of Wastewater Subjected to User Charges during Calculation Term

L : Total Pollution Load of Wastewater Subjected to User Charges during Calculation Term (for Each Item of Water Quality)

d. Calculation of user charges

In accordance with the bases for calculation, the user charges per cubic meter of ordinary and specified wastewater can be computed by combining the above-mentioned unit costs. In this case, the expenditures to be covered by the user charges for ordinary wastewater are restricted to the maintenance and operation cost, and the depreciation cost and the loan interest are excluded from the expenditures. The composition of the user charges computed in this manner is illustrated in Figure 3.4.

Fig. -3.4 Composition of User Charges


Cost for Quantity
Cost for Quality
Maintenance and Operation Cost
Repayment Cost of Facilities and Loan Interest
Repayment Cost of Facilities and Loan Interest
Maintenance and Operation Cost
(Yen/m³) Url(Ss–Sd)
(Yen/m³) Uml(Ss-Sd)
(Yen/m³) Urv
(Yen/m³) Umv
(Yen/m³) Url x So
(Yen/m³) Uml x So
(Yen/m³) Umv
(Yen/m³) Uml x So
Volume of Specified Wastewater Vs(m³)
Total Volume of Wastwater V (m³)
Volume of Ordinary Wastewater Vo (m³)
Water Quality Surcharges for Specified Wastewater
Water Quantity Charges for Specified Wastewater
Water Quantity Charges for Ordinary Wastewater

(Note) So : Average Quality of Ordinary Wastewater (ppm)
Ss : Quality of Specified Wastewater Subjected to Water Quality Surcharges (ppm)
S$_D$: Average Quality of Domestic Sewage (ppm) ($S_D \fallingdotseq So$)

e. Determination of user charges

At the determination of user charges, the following considerations should be added to the above-mentioned result of calculation.

1) Measures for Preceding Investment

Since sewage works are occupied their considerable part by a preceding investment at the initial stage of construction, it is necessary to lighten user charges by appropriating money of general account to the part of expenditures originated from the preceding investment.

2) Adoption of Progressive Charges System

Although progressive charges system has been explained at 3.2.1, c, it is necessary to examine a concrete way of its application. In this occasion, the average value of user charges under the progressive charges system conforms to the bases for calculation described in 3.2.2, and the steps and values of progressive charges should be so determined as to keep balance between the total income of user charges and the expenditures to be covered by user charges.

3) Same User Charges System for Same Administrative Area

Unit costs for user charges are varied for each sewage treatment area, but it is desirable for equalization of administration to apply same user charges system for same administrative area. However, in the case that it is reasonable in view of the actual conditions of the area to adopt an independent user charges system for the specified area such as a newly developed urban area, it is not always necessary to apply the same user charges system.

4. CONCLUSION

The outline of financing of sewage works and the actual conditions and the basic principle of sewerage user charges in Japan have been described as above. Financing of sewage works should be changed corresponding with the variety of the role of sewerage as stated at the preface. Particularly, because the role of sewerage in the aspect of water pollution control and preservation of water resources are increasingly expected today, proper maintenance and operation of sewerage facilities has become more and more important problem. For this reason, it is indispensable to seek sound finance in the aspect of management of sewage works, and proper application of sewerage user charges system is one of its requirement.

For instance, as to the principle of the cost imposition of tertiary treatment, it will become necessary to establish more detailed rule in accordance with the purpose of tertiary treatment, though it is recognized today that the cost for tertiary treatment shouldn't be covered by user charges excepting the cost for specified wastewater as stated at 3.2.1, d.

Furthermore, it is required to rescrutinize the range of specified wastewater the cost of which should be covered by user charges including the capital cost, in conformity with PPP.

The rule for sewage works financing compatible with the demand of new era is now being studied in the 4th Committee of Sewage Works Financing as mentioned in the preface. We wish that the result by the committee could be presented at the next conference.

CHAPTER 4
SEWER USER CHARGE SYSTEM IN OSAKA CITY

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1. INTRODUCTION

Sewarage is a fundamental facility in cities which protects urban areas from flood, improves the living environment by the diffusion of flush toilets and the prevention of the stagnation of waste water, and prevents the pollution of public water bodies. Various kinds of facilities that systematically constitute the sewerage; sewer, pumping stations and sewerage treatment plants, must display their effects to the fullest in an organic manner. The utility and benefit brought by sewerage are dual in nature, bringing both public and private assets to the people living in the community.

The construction of sewerage requires a great amount of money. However, sewerage construction itself is obviously not the objective. In other words, sewerage cannot achieve its true objective until it is completely maintained and managed. Consequently, expenses for sewerage easily surpass those for roads and rivers. This is the reason why the security of financial resources is the key to the execution of sewerage works and therefore is our important task to grapple with.

2. SEWERAGE WORKS AND FINANCE OF OSAKA CITY

2.1 PRESENT SITUATION OF SEWERAGE

The city of Osaka has a total area of about 21,000 ha, most part of which is lowlands made of sedimentary earth and sand of Yodo River and Yamato River and is below the flood level or the high tide level of Osaka Bay. Therefore, it is necessary to drain sewage with pumps for up to 90% of the city area. Besides, in addition to the above-mentioned two rivers, there are many other rivers flowing through the city, to which sewage is discharged. Its environmental water quality standard is set at 5 – 10 ppm of BOD value. The sewerage plan of Osaka City is shown in Fig. 1.

The construction of sewerage in Osaka City was commenced in 1894, prompted by the poor discharge and the prevalence of cholera. Through various kinds of development since then, the diffusion of sewerage was energetically promoted especially after the World War II. As a result, the diffusion rate has now reached 93%. The diffusion of sewerage and outline of facilities are shown in Table 1.

Combined sewerage system is adopted for discharging sewage and about 60% of the total pumping capacity of the pumping stations is dependent on Diesel pumps to provide for disasters. The sewage is treated by secondary treatment based on the activated sludge process (the primary treatment by plain sedimentation is being improved for secondary treatment) and the sludge is treated and disposed of by digestion, dewatering, incineration (construction is under way to incinerate all the sludge) and landfill.

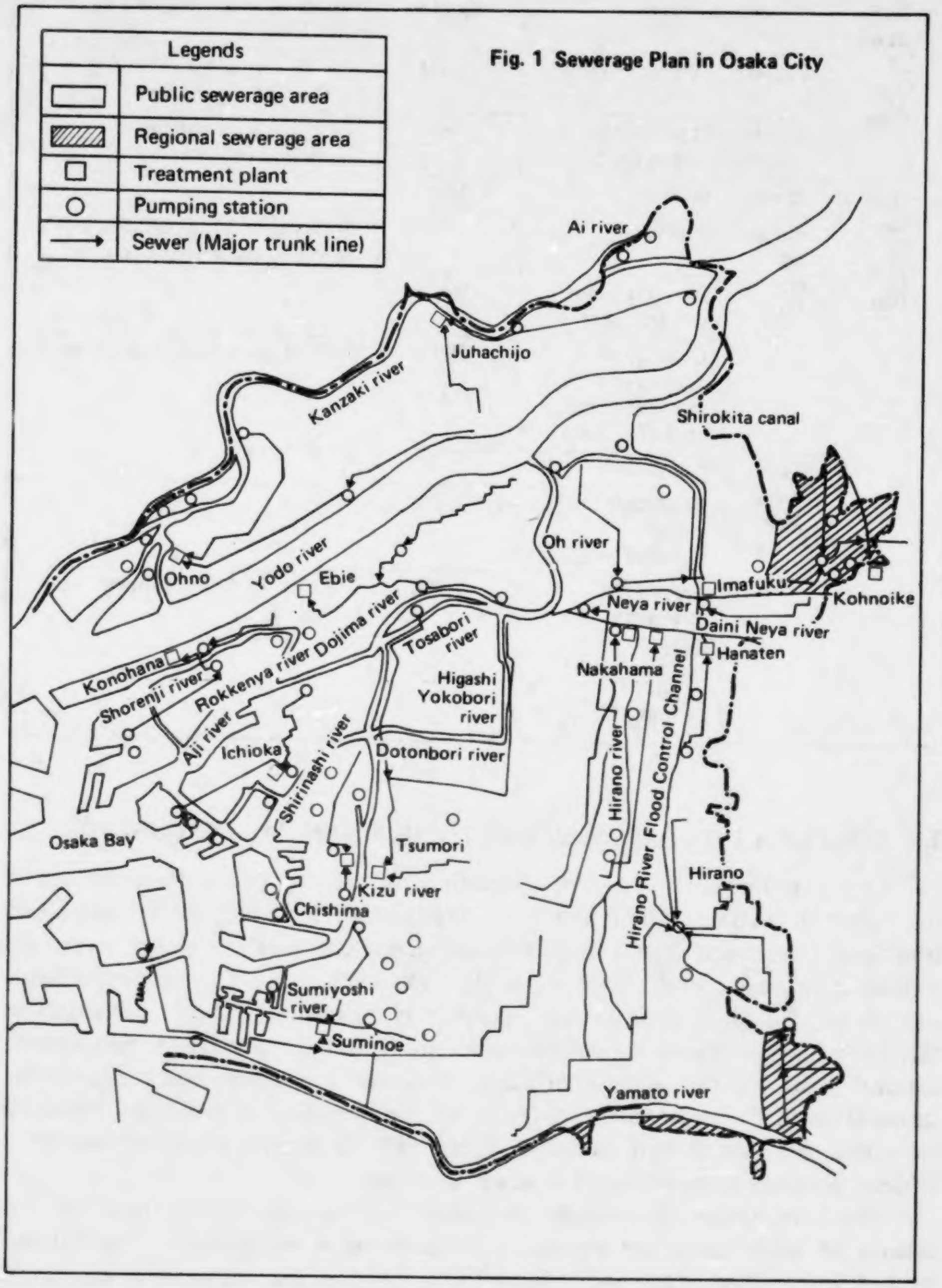


Fig. 1 Sewerage Plan in Osaka City

Table 1 Diffusion of Sewerage and the Outline of Facilities in Osaka City (As of the end of fiscal 1977)

Division				Remarks
Diffusion of sewerage	Sewered area (ha)		17,118	93.0% of total planned area of 18,410 ha
	Population served (1,000)		2,671	96.1% of total population of 2,779,000 as of today
	Number of households with flush toilets (1,000)		670	95.8% of total number of households of 700,000
Outline of facilities	Sewer	(km)	3,941	
	Pumping stations	Number	77	Inclusive of 11 pumping houses in sewage treatment plants
		Pumping capacity (m^3/sec)	899.5	
		Capacity of diesel pump included in the above (m^3/sec)	504.0	56% of the total pumping capacity
	Treatment plants	Number	12	
		Treatment capacity (1,000m^3/day)	2,570	
		Secondary treatment capacity included in the above (1,000m^3/day)	1,128	44% of the total treatment capacity

2.2 CONSTRUCTION EXPENSES AND THEIR FINANCIAL RESOURCES

In Japan, the construction of sewerage is financed by national expenditure in the name of national grant and local expenditure including prefectural grant, beneficial assessment, loan and general municipal expenses. Sewerage works are divided in two categories; subsidiray works and works financed by municipal funds and the breakdown of their financial resources is as shown in Table 2. According to this Table, up to 93% of the construction expenses of the sewerage is financed by national grant and the issuance of loans. However, the redemption funds for the issued loans that occupy 65% of the total financial resources in designated cities (10 big cities including Tokyo and Osaka) and 46% in general cities are causing a difficult problem in the financing of sewerage works.

The constitution of financial resources for sewerage construction and the balance of issued loans are shown in Tables 3 and 4, respectively. The balance reached ¥175 billion at the end of fiscal 1977, 1.5 times that for water works (¥120 billion).

Table 2 Breakdown of Financial Resources for the Construction of Public Sewerage in Japan

Division		Working expenses	National grant	Loans	General local funds
Designated cities	Subsidiary works	45%	28%	15%	2%
	Works financed by municipal funds	55	–	50	5
	Total	100	28	65	7
General cities	Subsidiary works	75	47	24	4
	Works financed by municipal funds	25	–	22	3
	Total	100	47	46	7
National average	Subsidiary works	60	38	19	3
	Works funanced by municipal funds	40	–	36	4
	Total	100	38	55	7

Table 3 Constitution of Financial Resources for Sewerage Construction in Osaka City
(Unit: 100 million yen)

Fiscal year	Working expenses	National grant		Issuance of loans		General local funds	
1966	72.2	11.4	16%	53.1	73%	7.7	11%
68	108.0	20.2	19	73.7	68	14.1	13
70	160.0	31.0	19	100.6	63	28.4	18
72	240.0	62.1	26	143.8	60	34.1	14
73	226.1	44.7	20	148.9	66	32.5	14
74	280.7	80.6	29	183.6	65	16.5	6
75	324.2	88.5	26	211.5	67	24.2	7
76	324.0	79.0	24	213.6	66	31.4	10
Total 1972 - 1976	1,395.0	354.9	25	901.4	65	138.7	10

Table 4 Transition of Balances of Issued Loans for Sewerage Construction in Osaka City
(Unit: 100 million yen)

Fiscal year	1966	1968	1970	1972	1973	1974	1975	1976
Balance	257	379	538	790	923	1,085	1,313	1,527

2.3 OPERATION AND MAINTENACE EXPENSES AND CAPITAL EXPENSES AND THEIR FINANCIAL RESOURCES

The operation and maintenance expenses and capital expenses for sewerage comprise (1) personnel expenses and supplies expenses required for the repair of sewer, dredging, the operation and maintenance of pumping stations and sewage treatment plants, the disposal of sludge generated in the treatment plants, the examination and regulation of effluent quality and the collection of sewer user charges (referred to as operation and maintenance expenses) and (2) depreciation expenses and interests for issued loans (referred to as capital expenses). Although efforts have been continuously made to minimize the operation and maintenance expenses and capital expenses, they have increased drastically mainly due to the diffusion of sewerage, the increase in the number of facilities (see Table 5), the rise in power rate, chemicals cost, sludge disposal cost and personnel expenses, as well as the ever-increasing amount of interests payable and depreciation expenses. This situation is well described in Table 6. The impression of the scale of these expences can rightly be made by the comparison with the 1978 budget of Osaka City. The operation and maintenance expenses and capital expenses for sewerage is ¥40 billion, or approximately 13% of the sum (¥320 billion) of the municipal tax revenue and the revenue from the local allocation tax. Besides, annual expenses amount to ¥15,000 per capita, or as high as 75% of the individual municipal tax (¥20,000) in Osaka City.

Table 5 Transition of Diffusion of Sewerage and the Outline of Facilities in Osaka City

Division			1960	1965	1966	1968	1970	1972	1973	1974	1975	1976
Diffusion of sewerage	Sewered area (ha)		3,426	6,155	6,729	9,903	11,074	14,153	14,929	15,551	16,196	16,464
	Population served (1,000)		–	1,183	1,387	1,932	2,131	2,362	2,533	2,605	2,615	2,640
	Number of households with flush toilets (1,000)		62	159	188	262	375	481	527	559	648	663
Outline of facilities	Sewer (km)		1,927	2,273	2,363	2,723	3,066	3,401	3,546	3,685	3,800	3,879
	Pumping stations	Number	71	78	78	78	68	66	69	70	76	77
		Pumping capacity (m^3/sec)	370	540	616	684	725	763	812	817	849	861
		Capacity of diesel pump included in the above (m^3/sec)	138	306	307	363	392	412	439	439	462	467
	Treatment plants	Number	3	7	7	10	11	12	12	12	12	12
		Treatment capacity (1,000m^3/day)	284	656	813	1,369	1,790	2,109	2,353	2,494	2,570	2,570
		Secondary treatment capacity included in the above (1,000m^3/day)	53	108	108	294	349	639	832	832	908	1,128

2.4 FINIACIAL SITUATION OF OSAKA CITY AND SEWERAGE WORKS

Since 1973, the financial situations of autonomies have been deteriorating with little expectation for the increase in local tax revenue and each autonomy has therefore been making serious efforts to curb its administrative expenses. Despite such efforts, the financing structure has been necessitating the increase in compulsory expenses such as personnel expenses, expenditures for welfare and other assistance and public bond expenses year after year, ever strengthening the rigidity of financing. Table 7, which describes the financial situation of Osaka City, shows the decrease in general financial resources including municipal tax that can be appropriated for investment and extraordinary expenses, as evidenced by the drop from 31% in 1970 to 10% in 1978.

Table 6 Transitions of Operation and Maintenance Expenses and Capital Expenses for Sewerage in Osaka City

(Unit: 100 million yen)

Division	1965	1966	1968	1970	1972	1973	1974	1975	1976
Personnel expenses	8.4	10.6	15.1	20.5	31.1	40.8	58.2	72.7	81.7
Supplies expenses	5.6	7.7	13.6	17.7	25.9	31.6	47.6	56.3	67.8
Sub-total	14.0	18.3	28.7	38.2	57.0	72.2	105.8	129.0	149.5
Interests payable	11.4	14.8	22.5	30.1	44.0	52.0	64.1	82.2	101.4
Depreciation expenses	8.5	9.3	13.8	20.7	28.5	35.0	38.6	46.8	54.5
Total	33.9	42.4	65.0	89.0	129.5	159.2	208.5	258.0	305.4

Table 7 Change in Financial Structure of Osaka City (General Account)

(Unit: 100 million yen)

Fiscal year / Division	Original budget for fiscal 1978	Settlement of accounts of fiscal 1974	Settlement of accounts of fiscal 1970
Tax revenue A	3,211	2,329	1,093
Ordinary expenses B	4,352	2,410	1,410
Specific funds for the above C	1,472	690	293
Taxes revenue covering ordinary expenses (B – C) D	2,880	1,720	749
Percentage of the above D/A	90%	74%	69%
Taxes revenue covering investment and extraordinary expenses (A – D) E	331	609	344
Percentage of the above E/A	10%	26%	31%

Table 8 Transition of Municipal Tax, General Municipal Expenses Appropriated for Sewerage and Standard Fiscal Demand

(Unit: 100 million yen)

Division	1965	1966	1968	1970	1972	1973	1974	1975	1976
Municipal taxes	468.7	520.9	690.7	960.4	1,249.3	1,587.6	2,018.7	2,075.9	2,247.4
Growth rate of the above	100	111	148	205	267	339	431	443	480
General municipal expenses appropriated for sewerage	18.2	26.9	43.5	59.6	87.4	95.2	114.0	130.4	150.2
Growth rate of the above	100	148	239	327	480	523	626	716	825
Standard fiscal demand	–	25.8	26.3	38.1	38.5	57.0	49.6	74.0	104.6

Despite such a background, the construction and improvement expenses and operation and maintenance expenses and capital expenses for sewerage works require a large amount of money. As a matter of fact, in the fiscal 1978 budget of Osaka City, a huge amount of ¥19.1 billion from general municipal expenses has been funneled into the fund for sewerage works. Table 8 shows the transition of municipal tax, general municipal expenses appropriated for sewerage works and standard fiscal demand. Even in the midst of the unprecedented crisis of the financial situation of the municipality, the growth rate of the appropriated general municipal expenses is much higher than that of the municipal tax. Furthermore, the appropriation of municipal tax is far larger than the necessary amount for the construction, maintenance and management of sewerage approved by the national government (standard fiscal demand).

2.5 NEW FIVE-YEAR PLAN FOR SEWERAGE WORKS

Based on the long history of sewerage works of well over 80 years, the Osaka Municipality has made strenuous efforts to maintain and manage every facility in order to achieve the objective of sewerage works. Now the diffusion rate is as high as 93%, flood is prevented in extensive areas, and the quality of river water in the city has been considerably improved. However, there are still many districts that are subjected to flood and the environmental standards for the quality of river water have not been achieved yet. At present, efforts are being made for the progress of the 5-year plan started in 1977 with the aim of improving sewerage in Osaka City. The contents of the 5-year plan follow;

1) Objective of the Plan

- Spread of secondary treatment in all the existing treatment plants and the introduction of tertiary treatment at Hirano treatment plant, with the aim of achieving and maintaining the environmental standards of water quality.

- Construction of new trunk sewer and additional pumping stations, with the aim of eliminating flood.
- Expansion of sewered areas to cover almost all city areas.
- Diffusion of flush toilets to almost all households.

2) Outline of the Plan

• Extension of installment of sewer	400km	(245km)
• Construction and expansion of pumping stations	25 places	(20 places)
• Expansion of treatment plants	12 places	(12 places)
Treatment capacity	2,960,000 m³/day	(2,696,000 m³/day)
Secondary treatment included in the above	2,960,000 m³/day	(2,696,000 m³/day)
Tertiary treatment included in the above	100,000 m³/day	(50,000 m³/day)
• Total working expenses	¥240 billion	(¥113.8 billion)

* In the parentheses are the figures for the first 3 years.

This 5-year plans is divided into the first phase (3 years from 1977 to 1979) and the second phase (2 years from 1980 and 1981), to be flexibly executed depending on the economic and financial situation of the times. The plan is now under way in compliance with the plan for revenue and expenditure (for the first three years) described in Table 9. In making this plan for revenue and expenditure, a lot of efforts were exerted to secure enough financial resources through the request for the increase in national grant, the security of general municipal funds and the revision of sewer user charges (double the previous charges in real terms). All this was done for the following reasons;

1) About two thirds of sewerage construction expeneses are financed by the issuance of loans. It is expected that an enormous amount of borrowings will be necessary in addition to the issuance of loans so far, which means that the interests payable will increase accordingly.
2) Construction of facilities results in the rise of depreciation expenses.
3) Due to the increased number of secondary treatment plants, the improvement of facilities, and the rise in prices and personnel expenses, the operation and maintenance expenses including power and sludge disposal expenses are on the rise.

In the following sections, sewer user charges that occupy about half of the revenue shall be described, centering on the current charges.

Table 9 New Plan for Revenue and Expenditure (1977-1979)

(Unit: 100 million yen)

1. Income balance (Management)			
Expenditure		Profit	
Operation and maintenance expenses	608	Sewer user charges	611
Interests payable	382	General account subsidy	577
Depreciation expenses	212	Other revenue	36
Cumulative deficit	58	Balance of comulative deficit	36
Total	1,260	Total	1,260
2. Capital balance (Construction and improvement)			
Expenditure		Revenue	
Construction and improvement expenses	1,138	National and prefectural grant	333
Redemption for corporate loans	121	Corporate loans	706
Repayment of general account borrowings	21	Reserve fund, etc.	246
Fund shortage	5		
Total	1,285	Total	1,285

3. SEWER USER CHARGES IN OSAKA CITY

3.1 TRANSITION OF SEWER USER CHARGE SYSTEM

Osaka City has a long history of sewerage and has long suffered the difficulty in securing financial resources for construction, maintenance and management of the sewerage. Osaka City is where the beneficial assessment system was introduced for the first time in Japan, although that system is prevalent nowadays all over the country. (It is not adopted at present because the document concerned were destroyed by fire and so on.)

Sewer user charge was also introduced in Osaka preceding its introduction in other cities. The study by Mr. Hajime Seki, the then Mayor of Osaka, published in 1928 evoked a hot controversy between the national government and the municipality over whether the sewer user charge should be borne by public or private expenses. The sewer user charge was finally approved after this argument. The approval of sewer user charge is based on the ground that "sewerage works can be pushed forward and the financing in compliance with the principle of fairness can be maintained only when people concerned are made to pay the majority of expenses for the construction, maintenance and management of sewerage in accordance with the degree of their use. And it is only natural to collect the charge for the use of sewerage as an establishment from the people who are no more in need of domestic sewage septic tanks with the completion of the municipal sewage treatment plant". The revenue that accrues from the collection of sewer user charges is to be appropriated for the financial resources for the operation and maintenance of sewerage and the redemption of public loans.

The transition of sewer user charge system in Osaka City is as shown in Table 10-1 and Table 10-2.

The year 1940 when the collection of sewer user charges first began was also memorable in that the sewage treatment plant was put into operation for the first time in Osaka City. In that year, diminishing rate system was adopted for the collection of sewer user charges. Since then until 1965, the unit price had been revised for several times while no change took place in the sewer user charge system itself. The meter-rate system was then introduced when the unit price was revised again in 1965.

In 1968, it was decided to prepare a 5-year plan for the improvement of sewerage with the objective of securing financial resources for sewerage works including operation and maintenance expenses and the redemption expenses of issued loans. By making a clear distinction between stormwater expenses to be borne by public expenses and waste water expenses to be subjected to the benefit principle, another revision of sewer user charges was made. This revision which linked the construction plan and sewer user charges was the first of its kind in Osaka City.

In 1972, the sewerage works plan was drastically expanded and revised in pursuit of the diffusion of sewerage in all city areas and the promotion of secondary treatment at sewage treatment plants. At this time, the diffusion rate of sewerage exceeded 60%, which in turn necessitated enormous amount of operation and maintenance expenses and redemption expenses for issued loans. This incident meant a significant turning point in the financing of sewerage in Osaka City. In pursuit of the adequate share of expenses and the observance of the Polluter Pays Principle, the water quality surcharge for various kinds of industries was established for the first time in Japan as a new sewerage charge system. For general users, the conventional meter-rate system was replaced by progressive rate system. Thus, a comprehensive revision of the system was carried out.

In 1974, four ranks were newly established in the conventional system in which uniform charge had been collected for the sewage exceeding 100m^3, thereby reinforcing the progressive rate system. Furthermore, another revision including the fractionation of ranks was made in 1977, with the consideration of domestic waste water and the adequacy of sewer user charges for big users.

Table 10-1 Transition of Sewer User Charge System in Osaka City

Kind			Unit water quantity	April 1 1940	April 1 1956
General sewer user charges	Minimum charge	For public bathhouse	Per month (up to $300m^3$)	yen	yen 600
		Other sewage	Exclusive use (up to $10m^3$)		30
			Common use (up to $10m^3$)		24
	For public bathhouse		per m^3	0.01	2
	For general use		up to $500m^3$ per m^3	0.35	3
			501 - $10{,}000m^3$ per m^3	0.20	2.40
			$10{,}001m^3$ or over per m^3	0.10	1.50
Sewer user charges for flush toilets	Faecal stool	Domestic use	per stool per month	0.18	(Nov. 1, 1951) 20
		Business use	per stool per month		40
	Urinal stool	Domestic use	per stool per month	0.18	10
		Business use	per stool per month		20

Kind		Unit water quantity	April 2, 1965	Oct. 1, 1968
Basic charges	For general use & public bathhouse	up to $8m^3$	30	50
		up to $10m^3$	40	70
	For common use	up to $8m^3$	24	24
Extra charges	For general use	$11m^3$ or over per m^3	5	10
	For public bathhouse	$11m^3$ or over per m^3	2.60	4.50
	For common use	$9m^3$ or over per m^3	3.90	3.90
Faecal stool	Domestic use	per stool per month	20	20
	Business use	per stool per month	40	40
Urinal stool	Domestic use	per stool per month	10	10
	Business use	per stool per month	20	20

Kind		Unit water quantity	Nov. 1, 1972
Basic charges	For general use & public bathhouse	up to $8m^3$	50
		up to $10m^3$	70
	For common use	up to $8m^3$	24
Extra charges	For general use	11 - $20m^3$ per m^3	10
		21 - $30m^3$ per m^3	15
		31 - $50m^3$ per m^3	16
		51 - $100m^3$ per m^3	17
		$101m^3$ or over	18
	For public bathhouse	$11m^3$ or over per m^3	4.50
	For common use	$9m^3$ or over per m^3	3.90
Faecal stool	Domestic use	per stool per month	Abolished
	Business use	per stool per month	
Urinal stool	Domestic use	per stool per month	
	Business use	per stool per month	

[Reference]

(1)	Diminishing rate system	from April 1940 to April 1965
(2)	Meter-rate system	from April 1965 to October 1972
(3)	Progressive rate system	from November 1972

(Note)

(1) Sewer user charges were revised eight times altogether from 1940 to 1965.
(2) The collection of flush toilet charges had been held in suspense until November 1951, the starting year of the collection. The flush toilet charge system was later abolished in November 1972.
(3) Industrial waste surcharge was put into effect on April 1, 1973.

Table 10-2 Transition of Sewer User Charge System in Osaka City

Kind		Nov. 1, 1974			April 1, 1977		
Basic charges	For general use & public bathhouse	 up to 8m³ up to 10m³	yen 50 70		 up to 8m³	yen 100	
	For common use	up to 8m³	24		up to 8m³	50	
Extra charges (per m³)	For general use	11 - 20m³ 21 - 30m³ 31 - 50m³ 51 - 100m³ 101 - 200m³ 201 - 500m³ 501 - 1,000m³ 1,001m³ or over	10 15 16 17 20 25 30 40		11 - 20m³ 21 - 30m³ 31 - 50m³ 51 - 100m³ 101 - 200m³ 201 - 500m³ 501 - 1,000m³ 1,001 - 5,000m³ 5,001 or over	20 27 30 35 40 50 60 80 90	
	For public bathhouse	11m³ or over	4.50		11m³ or over	5	
	For common use	9m³ or over	3,90		9m³ or over	10	
Water quality surcharge (per m³)	Division by water quality (ml/l)	April 1, 1973		Nov. 11, 1974		April 1, 1977	
		B.O.D (C O D)	S.S	B.O.D (C O D)	S.S	B.O.D (C O D)	S.S
	ppm ppm 201 - 300	yen 2	yen 2	yen 4	yen 5	yen 8	yen 9
	301 - 450	6	7	13	16	18	21
	451 - 600					30	35
	601 - 850	13	16	28	36	46	55
	851 - 1,100					66	79
	1,101 - 1,350	23	27	50	61	86	103
	1,351 - 1,600					106	127
	1,601 - 1,850	33	38	72	86	126	152
	1,851 - 2,100					146	176
	2,101 - 2,350	42	49	91	110	166	200
	2,351 - 2,600					186	225

3.2 PROVISIONS OF THE SEWERAGE LAW AND PROPOSAL OF THE THIRD COMMITTEE FOR THE INVESTIGATION OF SEWERAGE FINANCING

The provisions on sewer user charges and the rules on which the charges shall be determined are provided in the Sewerage Law. The Committee for the Investigation of Sewerage Financing has made some proposals with regard to sewer user charges. The contents of these are explained by Mr. Tamaki, the member of this conference, in his report.

3.3 PRINCIPLE OF SHARING EXPENSES

The Sewerage Law stipulates that the sewer user charges "should not exceed the proper sum of cost". But, what on earth is this cost?. Here the problem arises as to who should bear the expenses for what. Sewerage obviously has a public nature as it is a fundamental facility of cities. Its function is divided into the discharge of stormwater and the treatment of waste water, and its expenses into construction and improvement expenses and operation and maintenance expenses and capital expenses. Moreover, it inevitably involves various factors. For example, conditions of location vary, depending on the situation of each local autonomy. At the present stage, such a uniform principle is not yet established as to divide distinctively the portion to be borne by public expenses and that to be borne by private expenses. The latter should be the basis for the calculation of sewer user charges.

The first proposal made by the Committee for the Investigation of Sewerage Financing (in 1961) was instrumental in establishing the principle that both construction and management expenses concerning stormwater must be borne by public expenses and that those concerning waste water by private expenses. The second proposal (in 1966) recommended to expand the portion to be borne by public expenses, placing special emphasis on the importance of stormwater discharge function of cities. The third proposal (in 1973) suggested the principle that sewerage construction expenses be borne by public expenses and the operation and maintenance expenses concerning sewerage by private expenses, as the improvement of sewerage was regarded as a national task.

Referring to this proposal and the Polluter Pays Principle, the Osaka Municipality now follows the principle that operation and maintenance expenses and capital expenses regarding the discharge of stormwater be borne by public expenses and those regarding the treatment of waste water by private expenses.

However, there exists a problem concerning the share to be borne by public expenses proposed by the Third Committee. The third proposal meant to the effect that for specified waste water (discharged by big users such as buildings and factories), capital expenses for the said waste water must be included in the object of sewer user charges in addition to operation and maintenance expenses, and that the waste water falling short of the specified quantity must be borne by public expenses. Yet, the minimum quantity of this specified waste water in this proposal ($50m^3$/day) is too large in quantitative terms. The expense for the waste water in short of this quantity is not dealt with in the local allocation taxes. Nor is the share concerning stormwater satisfactorily calculated, as shown in Table 8. It is a hard fact that the present financial situation of the municipality can not bear such burden. Besides, as progressive rate system is adopted in the city's sewer user charge system, the charges for domestic waste water can be kept at a relatively low level. Under these circumstances, the Osaka Municipality did not accept the aforementioned proposal.

Although the necessity to attain the national minimum of the capital expenses borne by public expenses is well recognized, its range should be confined to domestic waste water (below 20 – 25m^3 per month). In the case of Osaka City, about 54% of total users discharge less than 20m^3 per month, and more than 76% discharge less than 30m^3 per month, as can be seen in Table 11. Therefore, it is considered possible to achieve the above-mentioned purpose.

Table 11 Number of Households and Amount of Effluent Classified by Ranks (Average of 3 years from 1977 to 1979)

Rank	Number of households		Amount of effluent	
		%	1,000m^3	%
10m^3 or less	270,782	26.3	21,423	4.3
11 – 20m^3	281,498	27.3	51,444	10.2
21 – 30m^3	233,352	22.6	69,116	13.8
31 – 50m^3	163,268	15.8	73,738	14.7
51 – 100m^3	50,416	4.9	40,295	8.0
101 – 500m^3	22,235	2.2	52,138	10.4
501 – 1,000m^3	2,561	0.3	21,393	4.3
1,001 – 5,000m^3	1,980	0.2	43,836	8.7
5,001m^3 or over	413	0.1	108,507	21.7
Public bathhouse	1,096	0.1	19,062	3.8
For common use	2,290	0.2	358	0.1
Total	1,029,891	100.0	501,310	100.0

3.4 CALCULATION OF EXPENSES SHARE RATE

The expenditure in the category of the income balance in Table 9 comes to an enormous amount of ¥120.2 billion for three years from 1977 to 1979 as mentioned in 2.3.

By clarifying the ratio of the portions of this necessary expenditure to be borne by public expenses and users, the amount to be borne by users shall be collected as sewer user charges. The concrete calculation method of this ratio is based upon the way of thinking described below and the result of the calculation is shown in Table 12. The percentage of share in total expenses is 45.2% for public expenses and 54.8% for users, or ¥54.3 billion and ¥65.9 billion respectively when converted into the sum of money. It must be noted that the expenses for tertiary treatment is not included as that specific treatment will not be put in operation during this period.

1) Operation and Maintenance Expenses

- Expenses for sewer (dredging expenses, etc.)
  Inorganic substance and others shall be classified as stormwater and waste water respectively, by the composition analysis of deposit in sewer.
- Pumping station expenses – To be shared in accordance with the ratio of pumping capacity for stormwater and waste water.
- Treatment plant expenses – Pumping station expenses shall apply to pumping facilities. Others shall be shared in accordance with the ratios of SS quantity, BOD quantity and water quantity.
- Water examination expenses and other expenses
  To be shared in accordance with the average share ratio of sewer expenses, pumping station expenses and treatment plant expenses. However, the expenses that can be divided clearly shall be shared accordingly.

2) Depreciation Expenses

- Sewer – To be shared according to the ratio of construction expenses in the case of separate sewerage system.
- Pumping stations – To be shared according to the ratio of the capacity of stormwater pump and waste water pump.
- Treatment plants – Most of the expenses is concerned with waste water.

3) Interests Payable

The same way of thinking as in the case of depreciation expenses applies to sewer, pumping stations and treatment plants.

Table 12 Breakdown and Share of Expenditure in Income Balance (for 3 years; 1977 – 1979)

(Unit: million yen)

Division	Total	Share			
		Municipal expenses		Sewer user charges	
Operation and maintenance expenses	60,871	21,869	35.9%	39,002	64.1%
Sewer expenses	14,671	10,635	72.5	4,036	27.5
Pumping station expenses	7,182	4,785	66.6	2,397	33.4
Treatment plant expenses	31,749	4,215	13.3	27,534	86.7
Water examination expenses	1,407	670	47.6	737	52.4
Other expenses	5,862	1,564	26.7	4,298	73.3
Depreciation expenses	21,171	11,000	52.0	10,171	48.0
Sewer	10,065	7,236	71.9	2,829	28.1
Pumping stations	2,442	2,121	86.9	321	13.1
Treatment plants	8,664	1,643	19.0	7,021	81.0
Interests payable	38,209	21,486	56.2	16,723	43.8
Sewer	23,013	16,545	71.9	6,469	28.1
Pumping stations	3,326	2,903	87.3	423	12.7
Treatment plants	11,869	2,038	17.2	9,831	82.8
Total	120,252	54,355	45.2%	65,897	54.8%

3.5 FUNDAMENTAL PHILOSOPHY UNDERLYING SEWER USER CHARGE SYSTEM

As sewer user charge is the charge for the use of facilities, it must be collected from the users in accordance with their individual use modes. The Sewerage Law stipulates that proper sum of cost under the efficient control should be obtained in calculating expenses and that sewer user charges should be determined in accordance with the amount of effluent and water quality with the aim of attaing fair share of burden among users.

Osaka City set the sewer user charges in accordance with the provisions of the Sewerage Law and the system is divided into general sewer user charges to be collected according to the amount of effluent and water quality surcharge to be collected according to the strength of water quality.

The general sewer user charges employ the progressive rate system. The adoption of this system, coupled with introduction of water quality surcharge, will definitely result in the decrease in the quantity of sewage discharged into sewer, the improvement of water quality, the saving and effective utilization of water resources and the recovery of resources. Moreover, it is rational in terms of cost.

The sewerage works are passive and cannot control actively the quantity of water that flows in. This is why the facilities are always equipped with maximum capacity. For this reason, 80% of the total expenses is occupied by the fixed expenses that is always required irrespective of the presence or absence of the discharge of waste water, such as basic power charge, personnel expenses, depreciation expenses for the facilities, interests payable, etc. This readily leads to the assumption that if the volume of the inflow of waste water into the sewer sould be the same every day, the highest efficiency of the expenses will be attained but that if the use of water varies due to the business and seasonal fluctuations, the efficiency will decrease as much.

The investigation into the actual situation of the use of water has revealed that the more water is used, the larger fluctuation of the amount of effluent. In this connection, this progressive rate system is appropriate in terms of cost.

3.6 COST OF GENERAL SEWER USER CHARGES

The comprehensive cost of expenses borne by sewer user charges (Table 12), that is, the expenditure in the income balance (Table 9) minus the expenses regarding stormwater borne by public expenses, is ¥45 per m^3. This is based on the calculation that the total amount of effluent for the three years amounts to about 1.5 billion m^3. Besides, excluding the amount to be collected as water quality surcharges that shall be described later, the amount that should be collected as general sewer user charges comes to ¥44 per m^3.

In calculating the cost in the case of progressive rate system, the ratio of required treatment capacity and the chargeable amount of effluent was figured out, dividing all the users in Osaka City in three groups in accordance with the amount of effluent.

- Group A - Users that discharge domestic waste water and small-scale commercial waste water with chargeable amount of effluent of 0 - 50 m³/month.
- Group B - Restaurants, hotels, etc. that use relatively large quantity of water with chargeable amount of effluent of 51 - 1,000 m³/month.
- Group C - Department stores, buildings, factories, etc. that use especially large quantity of water with the chargeable amount of effluent of more than 1,001 m³/month.

As a result, the ratio turned out to be 1.0 for Group A, 1.3 for Group B and 3.5 for Group C. This means that treatment capacity for 1 m³/day can yield the revenue for 1 m³/day in the case of users in Group A whereas fixed expenses required in the case of Group C will be 3.5 times as much as those for Group A.

The trial unit prices for each rank calculated on the basis of the abovementioned findings are as shown in Table 13.

The current sewer user charges system (see upper right column of Table 10-2) was established on this basis through the examination of the total amount of expenses in each group. To provide more information, the current charge systems for potable water and industrial water in Osaka City are shown in Table 14 and Table 15, respectively.

Table 13 Trial Unit Price Classified by Groups

(Yen/m³)

Group	Total amount of effluent for 3 years (1,000m³)	Transitory expenses	Fixed expenses	Total
A (0 – 50m³/month)	648,233	9.66	18.06	27.72
B (51 – 1,000m³/month)	347,258	9.66	23.48	33.14
C (Over 1,001m³/month)	508,438	9.66	63.22	72.88
Total	1,503,929	9.66	34.58	44.24

Table 14
The Current Potable Water Charges in Osaka City
(Put into effect in September, 1975)

Basic charges	Extra charges			
	Use	Per m³		
		m³	m³	Yen
¥230 up to 10m³	General use	11 –	20	40
		21 –	30	49
		31 –	40	58
		41 –	50	88
		51 –	100	108
		101 –	200	137
		201 –	500	155
		501 –	1,000	169
		1,001 –		180
	Business use	11 –	30	85
		31 –	50	130
		51 –		180
	For public bathhouse	11 –		28
¥130 up to 8m³	Common use	9 –		24

Table 15 The Current Charges of Industrial Water

(Put into effect in December, 1977)

Classification		Rate
Measured by mercury pressure difference meter	Contract quantity	¥27 per m^3
	When hourly consumption quantity exceeds hourly contract quantity (amount of water obtained by dividing contract quantity by 24 hours)	¥54 per m^3
Measured by meters other than the above	Contract quantity	¥27 per m^3
	In excess of contract quantity	¥54 per m^3

3.7 FUNDAMENTAL PHILOSOPHY UNDERLYING WATER QUALITY SURCHARGE

Two major merits of water quality surcharge are that (1) coupled with the progressive rate system of general charges, this system enables the recovery of cost through fair share of charges consented by users, and that (2) the system will work as an incentive for the users discharging sewage of high strength in their efforts to improve water quality, which will eventually facilitate adequate maintenance and management of sewerage.

In Osaka City, this specific system was introduced in 1972 and has been employed since then, as mentioned earlier. With BOD or COD and SS to be dealt with at sewage treatment plants as objects, it was decided that sewage having a strength over 200 ppm and a monthly discharge of more than 1,250m^3 be subjected to the collection of water quality surcharge for the following reasons;

- Provisions on water quality and those concerning proper sum of cost and the elimination of unfair discrimination were introduced with regard to sewer user charges when the Sewerage Law was revised.
- The responsiblities of enterprisers were specified in the Basic Law for Environmental Pollution Control and the Pollution Control Public Works Cost Allocation Law.
  The responsibilities of enterprisers to share the expenses of pollution preventive works in accordance of their benefit are strongly sought for.
- In order to realize the prevention of water pollution without delay, the improvement of sewerage and especially the promotion of secondary treatment at sewage treatment plants are necessary. This will, however, boost the operation and maintenance expenses and capital expenses, which will in turn widen the gap between the unit prices for treatment of high strength waste water and domestic waste water in the above-mentioned expenses.

It goes without saying that such harmful substances as to damage or hamper the functions of sewerage system shall not be discharged into the sewer until they are disposed of by pretreatment facilities to meet the regulation values specified in the Sewerage Law.

1) BOD or COD and SS are considered as objects of water quality surcharge on the following grounds;

- BOD or COD and SS are essentially the items that must be dealt with at sewage treatment plants. Although it is possible to treat them even if they are of rather high strength, the treatment becomes comparatively expensive. From the standpoint of fair share of burden, water quality surcharge was adopted.
- However, as effluent of very high strength causes an adverse effect on sewerage system, pretreatment facilities must be installed for such effluent. Though the maximum strength is set at 2,600 ppm, approval is required for water quality of 600 ppm of more.

2) Waste water of 200 ppm or more are subjected to the collection of water quality surcharge for the following reasons;

- In general, the water quality of domestic sewage is 120 - 180 ppm, the maximum being approximately 200 ppm.
- The regulated values in the Water Pollution Control Law are set at 160 ppm BOD or COD (daily average 120 ppm) and 200 ppm SS (daily average 150 ppm).
- Design criteria of sewage treatment plants are at 200 ppm, as a general rule.

3) Factories with monthly discharge of 1,250m^3 or more are subjected to the collection for the following reasons;

- Factories with monthly discharge of 1,250m^3 or more are relatively large-sized. Most of the small enterprises can, therefore, be excluded from the object of collection.
- The Sewerage Law and the Enforcement Ordinance of the said law stipulate that factories with daily discharge of 50m^3 or more be required to give notice of both quantity and quality of their effluent.
- Possible effects on sewerage system and the bearing on business routine were also taken into consideration.

4) The reason why the unit prices were set, classified by water quality (BOD or COD and SS) is as follows;

- The relation between the object items of water quality surcharge and unit prices can be clarified.
- Sewer user charge system becomes easy to understand.
  It also conforms to the philosophy underlying Article 20 of the Sewerage Law.
- Object items of water quality surcharge can be readily distinguished from other restriction items.
  Consequently, the contents of guidance for the installation of pretreatment facilities can also be clarified.
- The cost accounting of water quality surcharge was performed for each water quality item.

3.8 COST OF WATER QUALITY SURCHARGE

The expenses borne by sewer user charges as described in 3.4 and 3.6 can be divided into those concerning amount of effluent (water quantity expenses) and those concerning quality of effluent (water quality expenses). ¥35.3 billion is planned for the former and ¥36.4 billion for the latter for the three years from 1977 to 1979.

As a result of the ananlysis of the water quality expenses, ¥8.01 and ¥9.70 were obtained for BOD or COD and SS, respectively, per m^3 of inflow quantity and 100 ppm water quality. Sewer user charge was established for each rank based on these figures as shown in Table 10-2 (lower right part). The analysis method shall be described later.

It should be noted that only ¥1.8 billion is estimated as the revenue from water quality surcharge, a meager amount accounting only for 3% of total revenue from sewer user charges, which is estimated to reach ¥61.1 billion.

1) Analysis of Expenses Borne by Sewer User Charges into Water Quantity Expenses and Water Quality Expenses

The analysis is based upon the following;

- Water Quantity Expenses -
  Expenses for pumping stations, primary settling tanks and chlorine installations
- BOD or COD Expenses
  Expenses for aeration tanks and final settling tanks
- SS Expenses
  Expenses for sludge treatment facilities

2) Calculation Method of water quality Surcharge

The total amount of effluent to be treated for the three years from 1977 to 1979 is about 1.5 billion m^3. The water quality expenses thereof is ¥36.4 billion, which is analyzed into BOD (or COD) expenses (¥15.8 billion), SS expenses (¥20.3 billion) and water examination expenses (¥300 million). By dividing the BOD or COD and SS expenses by the load of inflow quantity and quality, unit price per m^3 (quantity) per 100 ppm (quality) can be figured out. These figures will then be multiplied by the difference (actual strength in ppm figure minus 200 ppm) to make them unit prices for individual ranks. On the other hand, water examination expenses shall be divided by the object amount of effluent of water quality surcharge (83,000 m^3) to obtain unit price per m^3. In this case, a fixed amount shall be added irrespective of strength.

4. RECENT TREND IN SEWER USER CHARGES

4.1 CHANGE IN TOTAL AMOUNT OF EFFLUENT

As described so far, Osaka City adopted the progressive rate system and water quality surcharge (applied in 1973) in its sewer user charge system in 1972. Later in 1974 and 1977, the revisions of unit prices and the reinforcement of progressive system were made. The change in the amount of effluent over years since 1973 is shown in two versions; classified by ranks (Fig. 2) and by sources (Fig. 3). The condition of monthly change is shown, classified by ranks (Fig. 4) and by sources (Fig. 5).

The total amount of effluent during this time period was 523 million m^3 (100)

in 1973, 498 million m³ (95) in 1974, 507 million m³ (97) in 1975, 496 million m³ (95) in 1976 and 492 million m³ (94) in 1977, showing a slight decline over years.

The classification by ranks is characterized by a rising trend in the case of users with small amount while in the rank of large amount and especially in the rank of 10,001 m³/month or more, the figure represents a drastic decline, 1977's figure only being 63% of that in 1973. The classification by sources reveals that the potable water quantity is flattening out whereas the quantity of industrial water and well and river water which markedly affect sewer user charges almost halved to 62% and 59% respectively. (See Table 17).

4.2 CHANGE IN QUANTITY AND QUALITY OF EFFLUENT SUBJECTED TO WATER QUALITY SURCHARGE

The monthly changes in the number of factories subjected to the collection of water quality surcharge and their amount of effluent are shown in Fig. 6. The yearly change in the amount of effluent and the change in load, both classified by object items, are shown in Figs. 7 and 8, respectively.

In 1977, amount of effluent decreased to 61% in terms of BOD or COD and to 31% in terms of SS, compared to the figure in 1973. Similarly, the load decreased to 51% in terms of BOD or COD and to 27% in terms of SS by the same comparison. The rate of decrease is rather high and worthy of special attention in the case of SS.

Is it because that SS can be more easily eliminated than BOD? In this general trend of decrease, high and low ranks indicated decreasing trend, whereas the intermediate ranks showed an increasing trend.

The shift in the average unit price in the water quality surcharge after the establishment and revisions of the water quality surcharge is shown in Table 16. Although the change in the unit price after the second revision is small, it marked a 7% decrease one year after the establishment and a 25% decrease one year after the first revision.

It is assumed that the water saving resulted in the drop of effluent amount to lower rank and that efforts had been made in pursuit of the improvement of effluent quality.

Table 16 Transistion of Average Unit Price of Water Quality Surcharges

Establishment and revisions	Number of months passed after revision	1	2	3	4	5	6	7	8	9	10	11	12
Establishment (April 1973)	Average unit price (Yen/m³)	8.9	8.9	8.8	10.7	9.4	9.4	9.7	9.0	8.4	8.5	8.9	8.3
	Index	100	100	99	120	106	106	109	101	94	96	100	93
First revision (Nov. 1974)	Average unit price (Yen/m³)	15.8	15.2	16.1	12.5	13.7	12.5	12.3	12.2	11.6	12.3	11.9	11.9
	Index	100	96	102	79	87	79	78	77	73	78	75	75
Second revision (April 1977)	Average unit price (Yen/m³)	26.6	26.6	25.1	24.8	25.2	25.8	25.5	24.8	25.8	26.1	30.2	31.5
	Index	100	100	94	93	95	97	96	93	97	98	114	118

4.3 DISCUSSION

It is somewhat rash to conclude that all these phenomena have been brought by the sewer user charge system. Many other factors such as the change in economic environment must be taken into account.

Related to not only factories but also activities of business offices in general, these factors are intricately entangled, permitting no easy grasp of the situation. As a trial, we examined the relation between the growth rate of industrial shipment in the city (adjusted by price index) and the growth rate of sewered area as well as the fluctuation in total amount of effluent and quantity and quality of effluent subjected to water quality surcharges. (See Table 17.)

Table 17 Rate of Increase/Decrease over Years of Amount of Effluent, Load, Industrial Shipment and Sewered Areas

(1973 = 100)

Classification			1970	1971	1972	1973	1974	1975	1976	1977
Total amount of effluent	By ranks	0 – 50m^3/day	98	104	92	100	105	111	115	119
		51 – 1,000m^3/day	72	73	102	100	91	95	91	93
		1,001 – 10,000 m^3/day	180	185	104	100	90	94	91	91
		Over 10,001 m^3/day	37	37	93	100	89	81	72	63
		Total	88	91	97	100	95	97	95	94
	By sources	Potable water	90	93	98	100	97	101	100	102
		Well and river water	87	91	88	100	88	85	72	59
		Industrial water	74	77	93	100	85	77	72	62
		Total	88	91	97	100	95	97	95	94
Object of water quality surcharges	Amount of effluent	BOD or COD				100	87	73	67	61
		SS				100	63	47	44	35
	Load	BOD or COD				100	78	58	56	51
		SS				100	61	32	39	27
Sewered area			83	84	90	100	105	109	113	115
Industrial shipment			89	88	94	100	89	82	84	

According to this Table;

- The sewered area is much larger in 1976 than in 1973.
  (An increase of 13%. However, the rate of increase in total amount of effluent is not so high.)
- Production decreased at a little higher rate (–16%).
- Total amount of effeuent showed a slight decline (–5%).

As many conditions overlap, it is not at all easy to make a conclusion. But it is suspected that slackened business activities had something to do with the decline in amount of effluent. Whatever the reason may be, the fact is that the amount of effluent from big users and that of well and river water and industrial water recorded a big decline.

Worhty of special note is the radical drop in the quantity and quality of effluent related to industrial waste surcharges. These phenomena seem to be the effect of this sewer user charge system.

As many factors such as seasonal fluctuations as shown in Figs. 4 and 5 exist, more detailed analyses and examinations will be necessary. And to what extent the amount of effluent will decrease in the future is an interesting problem.

The examples shown in Fig. 9 describe the efforts to reduce the amount and load of effluent at factories.

It can be easily assumed that the total amount of effluent decreased somewhat due to the decreased production caused by the oil shock. However, the amount of effluent was, in fact, reduced to a larger extent and the quality of effluent was improved likewise.

To cope with this declining trend in the amount of effluent, the following things are reckoned to be of fundamental importance;

1) Review of the basic discharge per unit of production set at the time of planning of facilities

   It is necessary to check the basic discharge per unit of production (especially of big users) under the present conditions of the existing facilities, and seek for the ideal discharge per unit of production (water saving type) which is most suitable for resources and energy saving policies and sewer user charge system.

2) Execution of phased expansion of facilities and especially of treatment facilities

   Keeping an eye on the actual increase in the quantity of inflow, it is necessary to expand the facilities step by step, though such construction work might be time-consuming.

This phased execution scheme is generally employed for the construction of treatment plants. The same policy will be applied to the future expansion plan of sewerage in Osaka City.

The municipality is determined to set fixed expenses (construction work expenses) which is definitely necessary, grasp accurately the amount of effluent, and establish proper sewer user charges, thus securing financial resources.

5. CONCLUSION

The sewerage in Osaka City has been continuously constructed over the past 80 years in order to prevent the pollution of public water bodies and to improve the urban environment. At the same time, the established facilities are given as much maintenance and management as possible to achieve the objectives of the sewerage system. Throughout this development, people concerned have been faced with administrative and technical problems as well as financial problems and have suffered the difficulty in securing financial resources for construction, maintenance and management. Today, efforts are being made to ask the national government to secure financial resources for construction whereas part of municipal tax and sewer user charges constitute the financial resources for maintenance and management.

Observing the Polluter Pays Principle, the sewer user charges adopt both progressive rate system and water quality surcharge with proper and reasonable cost determined in accordance with the use mode including the quantity and quality of sewage.

This charge system works as an incentive for the reduction of amount and the improvement of quality of effluent, which then leads to the saving and effective utilization of water resources, the recovery of resources, and the efficient construction, maintenance and management of sewerage.

However, the sewerage in Osaka City is not free of problems. Even with regard to sewer user charges alone, many problems concerning the increase in the national grant for construction, the improvement of loan issuing conditions and the increase in the local allocation tax could be cited. Besides, to reduce the operation and maintenance expenses, it is necessary to develop better technology for water treatment and sludge treatment and disposal. Furthermore, the increase in the share of expenses required for the construction, maintenance and management of tertiary treatment facilities to be borne by the national government is strongly sought for.

Though the financing problem of sewerage has long been existing, it is also a new problem. We intend to resolve this problem for the sake of better management of sewerage facilities. The other day, the Fourth Committee for Investigation of Sewerage Financing was inaugurated to begin examining what the new sewerage financing should be under the new circumstances. And again, we are harboring so many expectations for the outcome of this meeting.

Fig. 2 Yearly Change in Amount of Effluent by Ranks

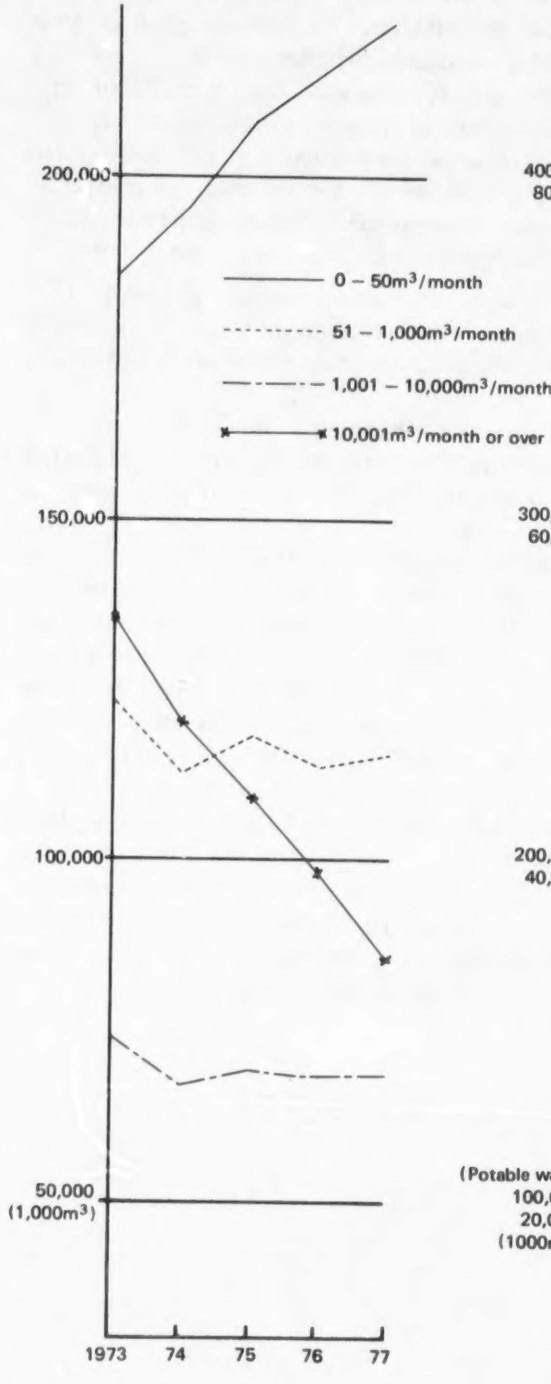


Fig. 3 Yearly Change in Amount of Effluent by Sources

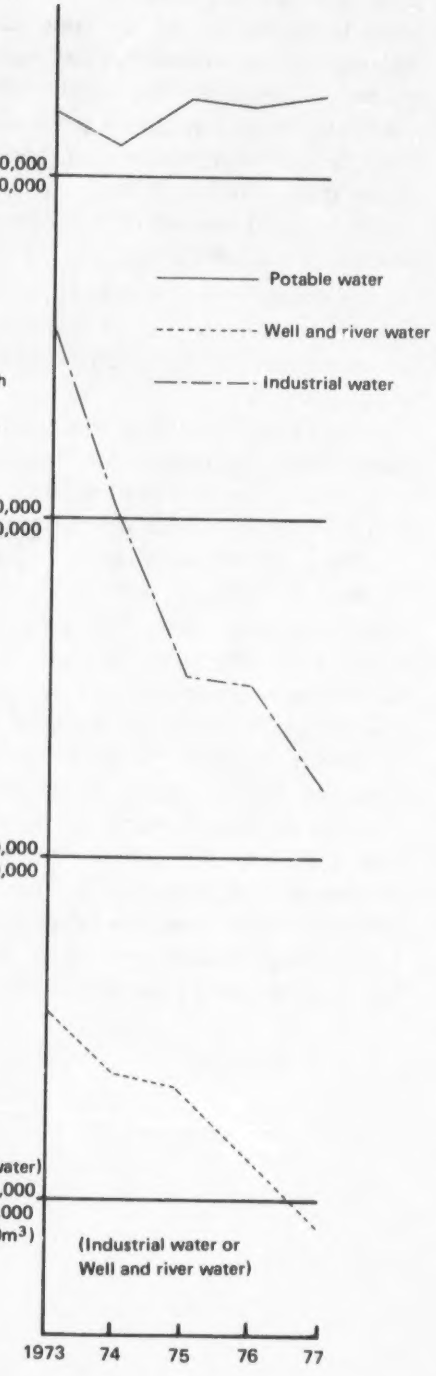


Fig. 4 Monthly Change in Amount of Effluent by Ranks

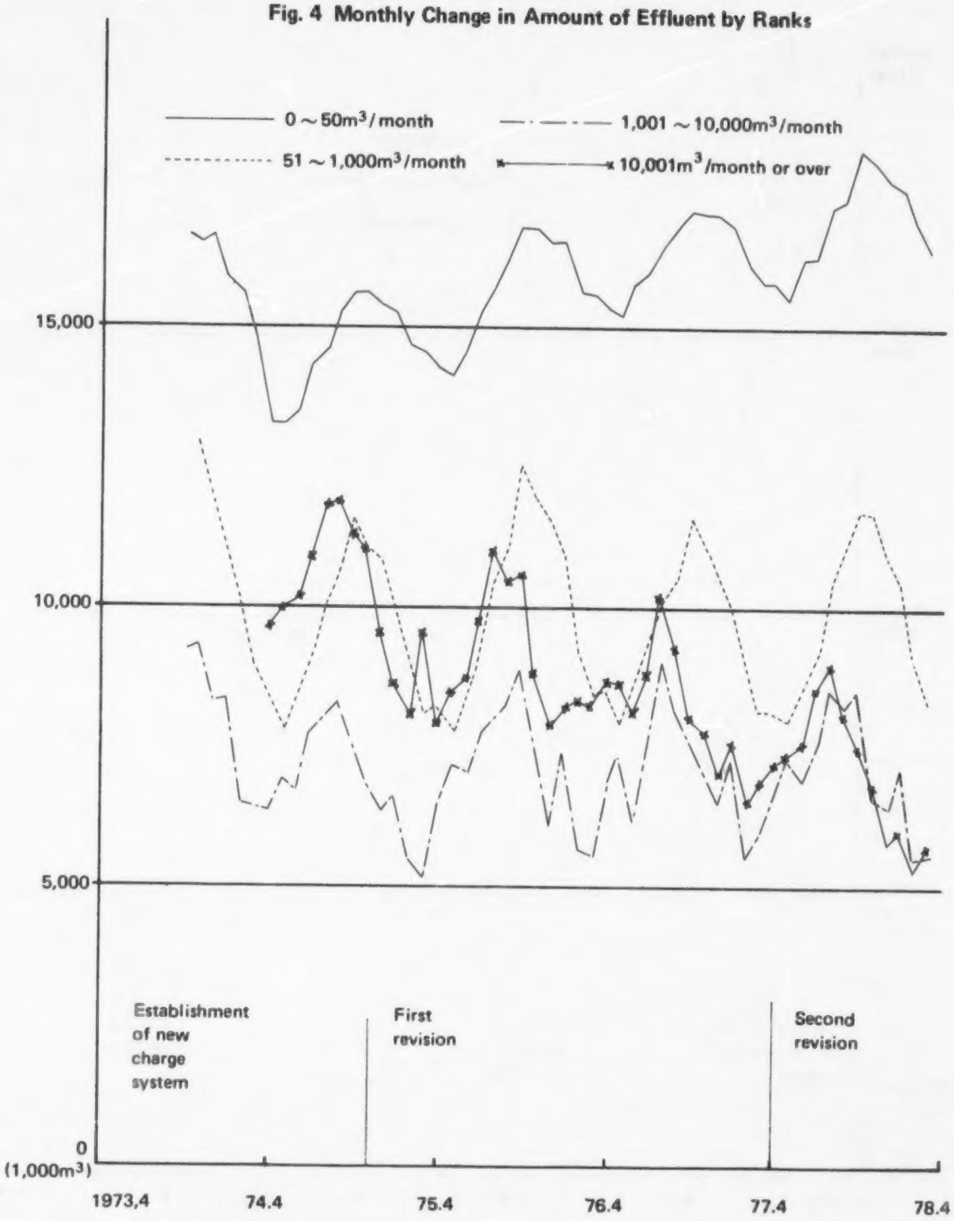


Fig. 5 Monthly Change in the Amount of Effluent by Sources

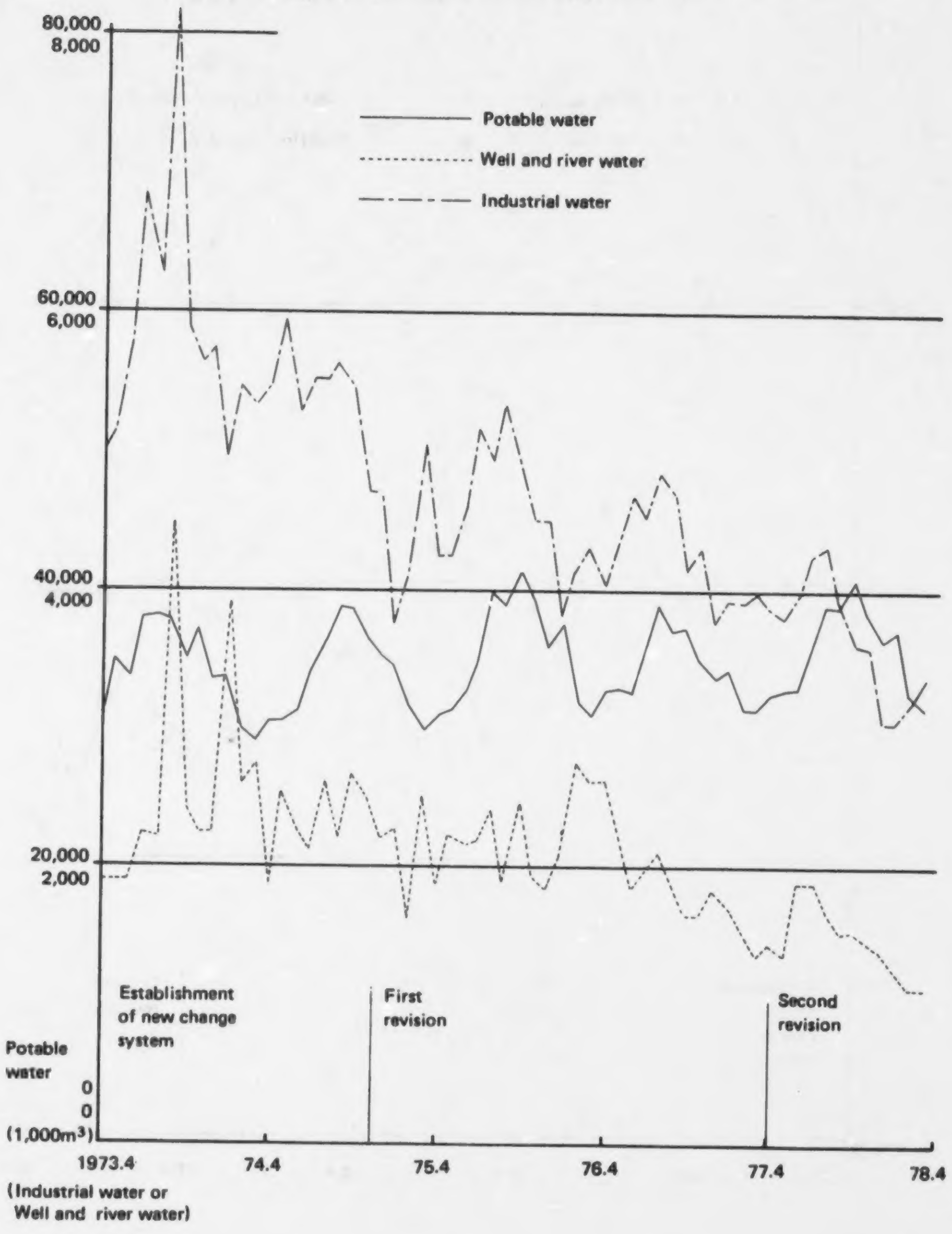


Fig. 6 Monthly Change in the Number of Factories Subjected to Water Quality Surcharges and their Amount of Effluent

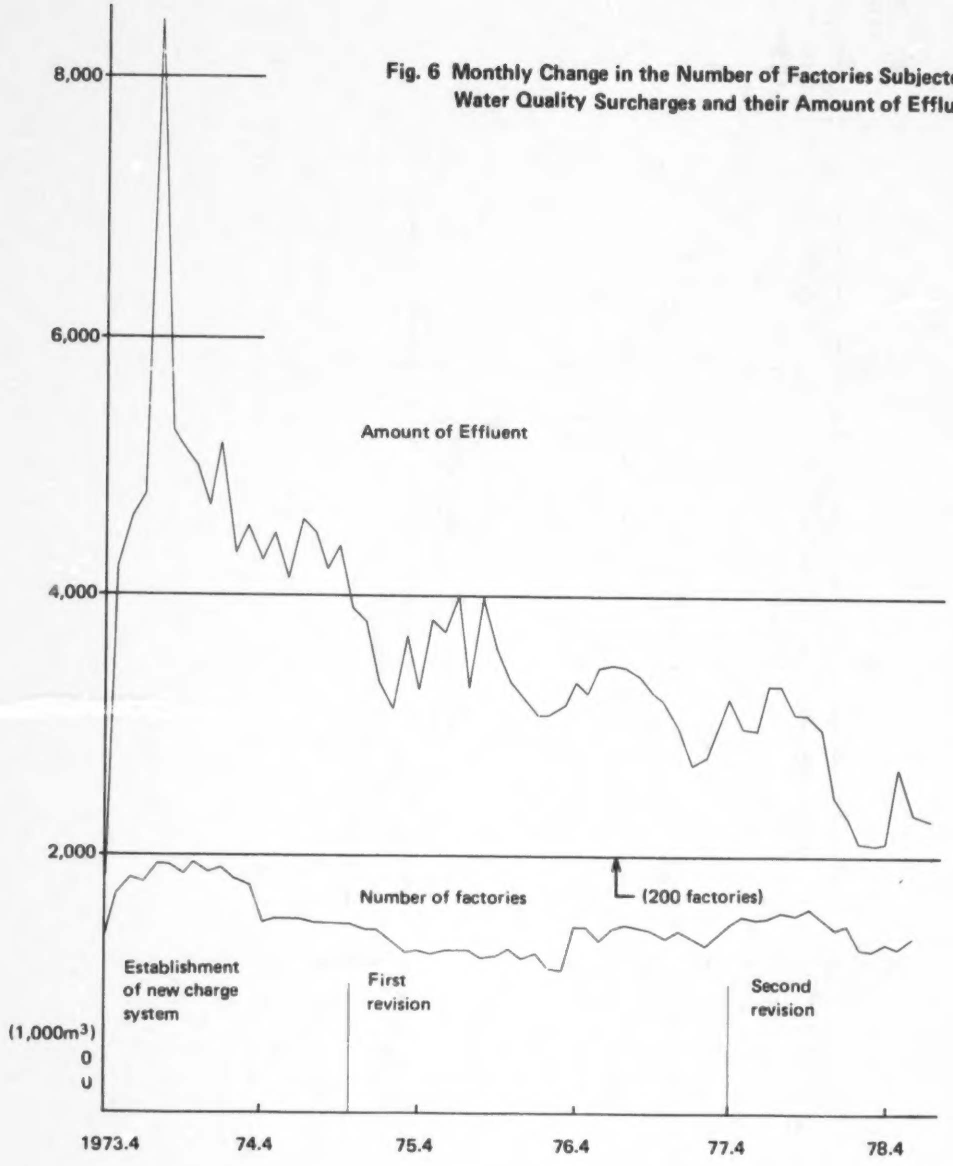


Fig. 7 Yearly Change in Amount of Effluent by Object Items of Water Quality Surcharges

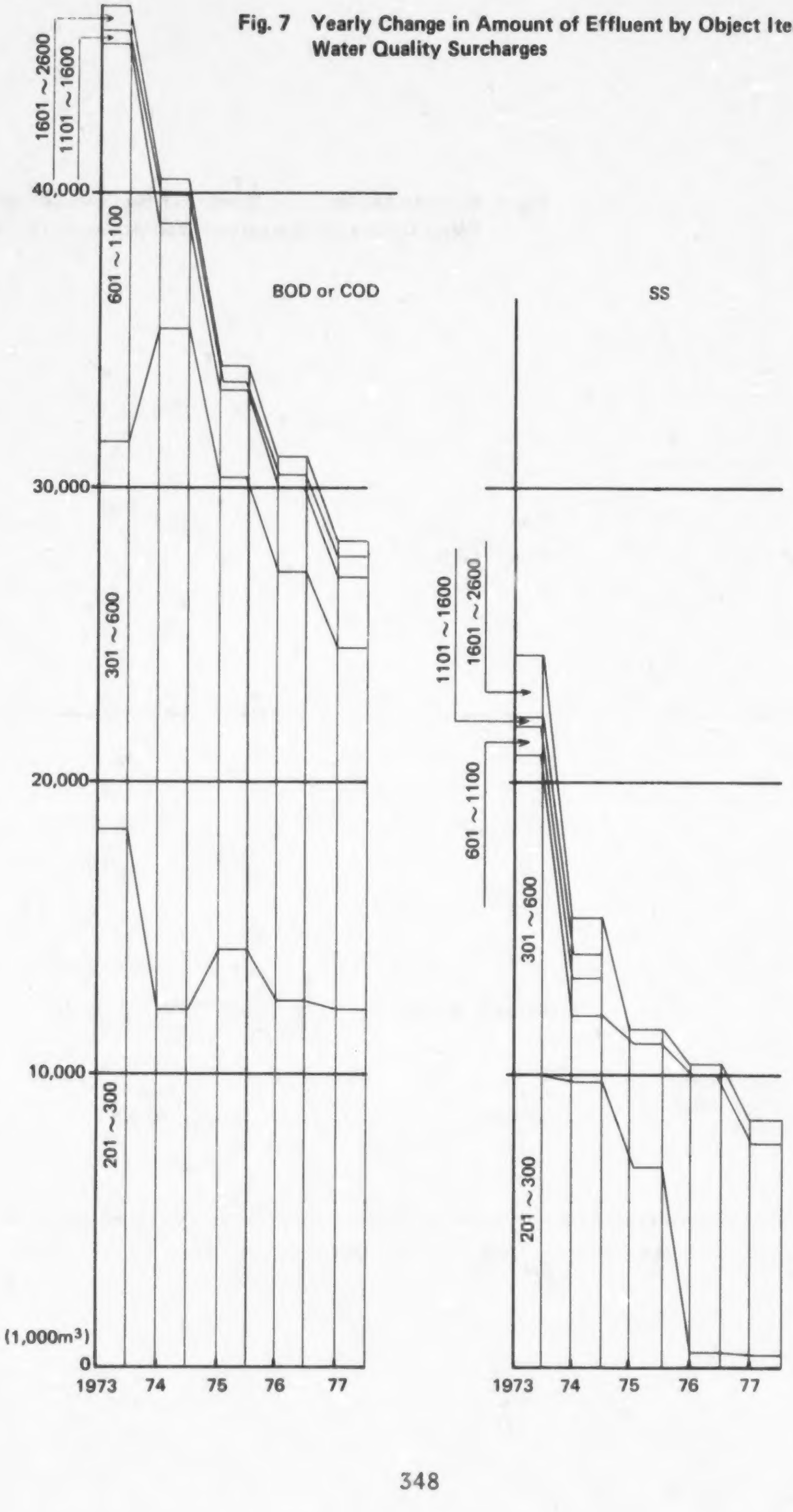


Fig. 8 Yearly Change in Load by Object Items of Water Quality Surcharges

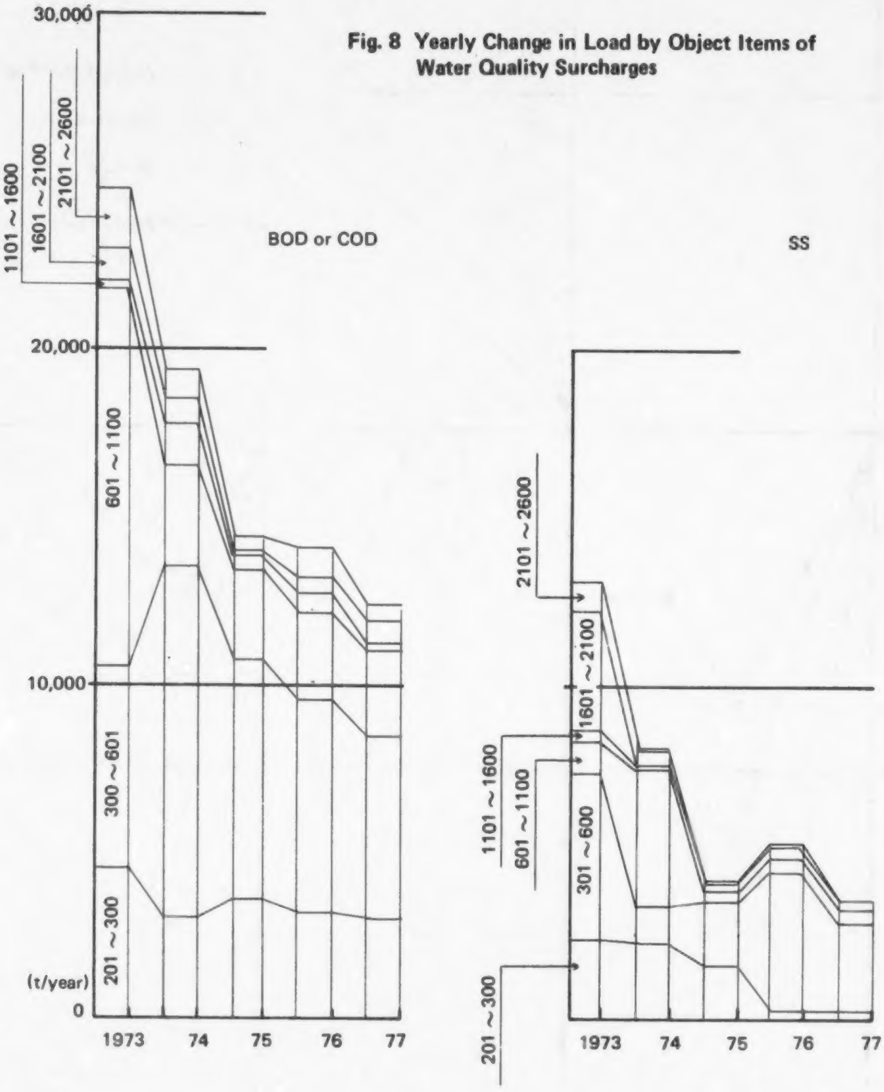
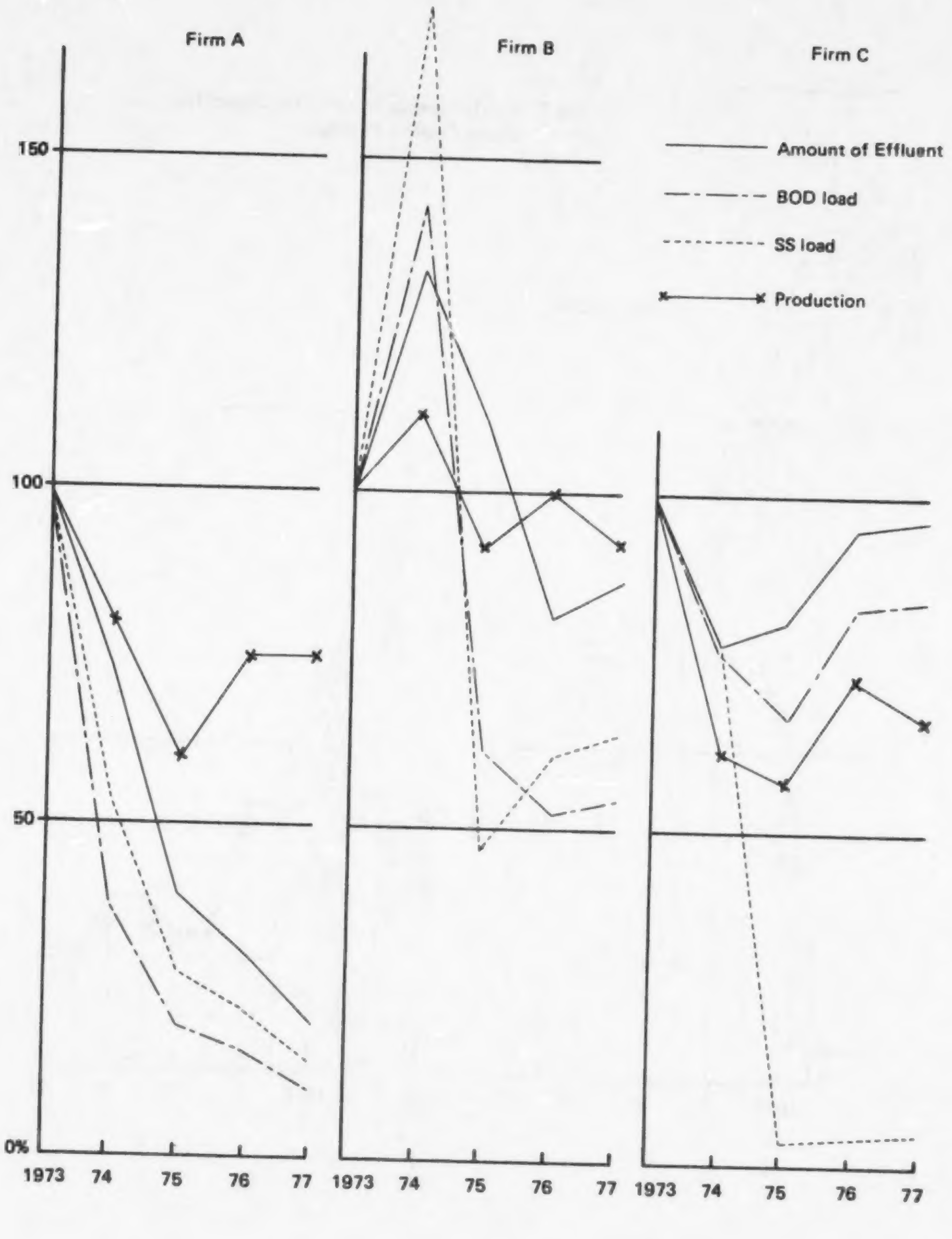


Fig. 9 Example of Yearly Change in Amount of Effluent and Load (1973 = 100)



CHAPTER 5

New Supervision and Control System of Sewage Treatment Plant in Yokohama

– Effective application of microcomputer –

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Introduction

The municipality of Yokohama has taken up the subject of its sewage projects in almost all areas of the City, and plans the construction of sewage treatment plants in eleven locations.

In the 16 years since 1962, when the first sewage treatment plant in The Chubu district (Chubu S.T.P.) was put into operation, plants in The Nanbu, Hokubu, Totsuka 2nd, Kohoku, Midori, and Kanagawa districts have been constructed, in that order, and the seven plants currently are being operated.

The operational system of a sewage treatment plant is based upon the trend toward centralized supervision and automatic control, but the details have changed with the lapse of time, keeping pace with the innovations of technology and development of the equipment.

First, there was a centralized supervision and control system by blocks, whereby the total process was divided into several blocks. However, thereafter a centralized supervision and control system, where not only the total process of the plant was consolidated but also the remote supervision and control of the relaying pumping stations was included, became available, and this new system has been strengthened in the sixth plant in the Midori district. The seventh plant, in the Kanagawa district, has the greatest treatment capacity in the City (572,000 m^3/day), and for this plant, a centralized supervision and distributed control system has been newly applied, based upon our experience and the rapid progress made in the instrumentation and data processing devices, and also on the premise of possible technological developments, and the complications and expansion of the subject to be controlled.

This paper is involved with the progress of the supervision and control systems in the sewage treatment plants of the City as shown in Fig 1, and an outline of the centralized supervision and distributed control systems of the Kanagawa Sewage Treatment Plant.

1. Centralized supervision and control system

The purpose of centralized supervision and control is to concentrate and automate, as far as possible, monitoring of the processing conditions, process control, and gathering and recording of the process data, to attain stable processing, efficient operation of the facilities, man power saving, energy saving, improvement of the work environment and a reduction of treatment costs.

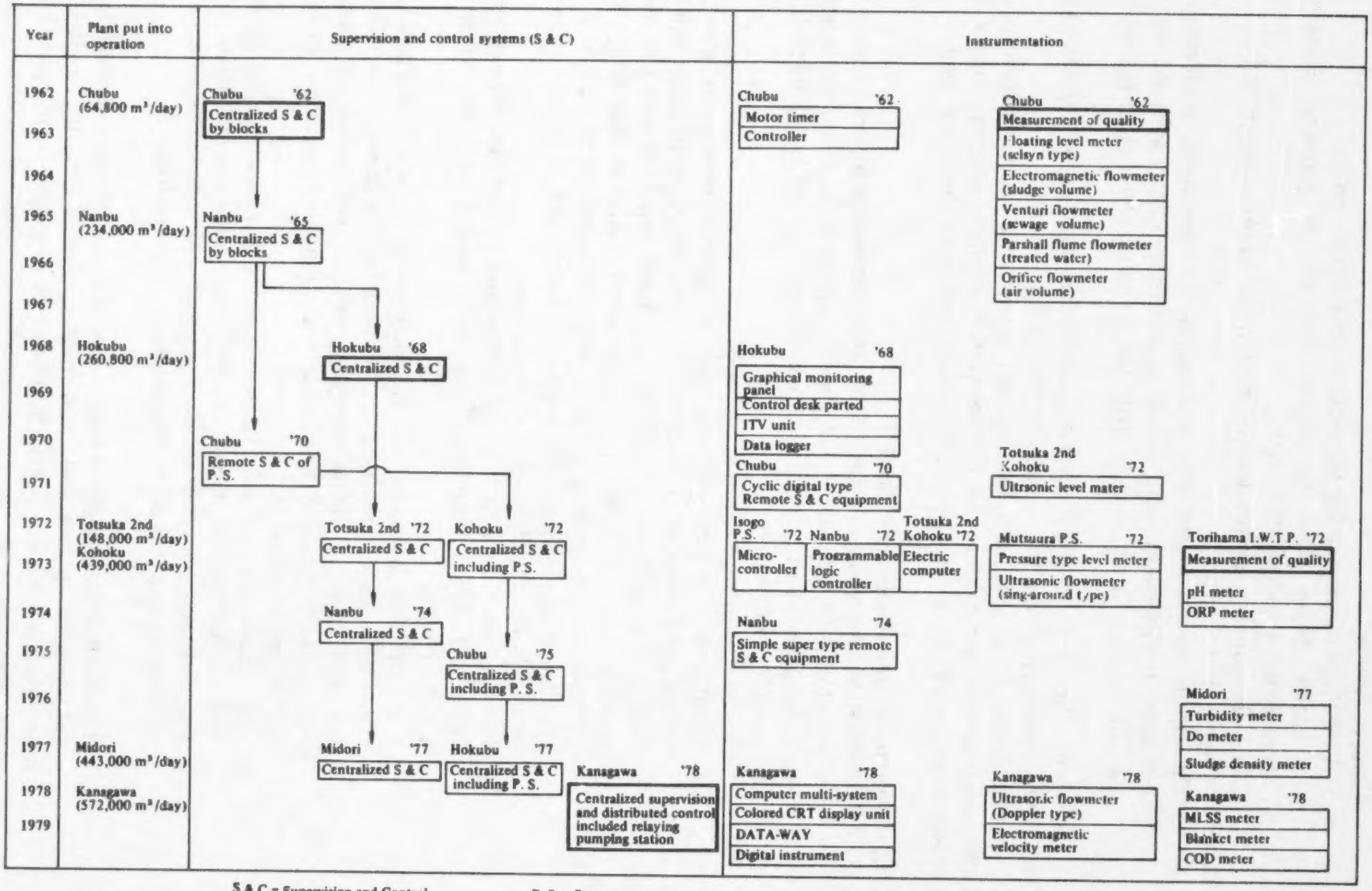
1.1 Centralized supervision and control by blocks

For control of the Chubu S.T.P. which was put into operation for the first time in the City, the latest technology for automatic control at that time were applied. The outline was as follows:

a) A centralized supervision and control system by three separate blocks was applied: a sewage treatment block, sludge transfer block (relevant to return, excess and concentrated sludge) and a sludge treatment block (as the treatment of sludge mixed with dipped-up night soil was conducted under special condition in this country, this process was under the control of the other bureau).

b) In the sewage treatment block, centralized supervison and manipulation conducted from a central operation room and automatic operation were

Fig. 1 Progress of Supervision and Control System of Sewage Treatment Plant in Yokohama City



S & C = Supervision and Control

P. S. = Pumping station

I.W.T.P. = Industrial Wastewater Treatment Plant adopted for grit chamber equipment, main pumps, blowers, etc.

Control devices:

- A regulator for controlling to keep the air volume constant
- A motor driven timer to control time for the primary, excess and concentrated sludge withdrawal
- A logic sequence for interlocking motions of the main pumps and mechanical screens

c) The measuring instrument was used mainly for quantitative measurement. For water level measurement, a float system (selsyn type) was used, and for the flow rate measurement, a Parshall flume, venturi meter etc. were used.

In 1965, the monitoring and control of the sludge transfer block in the Chubu S.T.P. was consolidated into the central operation room.

The Nanbu S.T.P. was put into operation in the same year, and, in this plant also, a centralized supervision and control system by three separate blocks, i.e., sewage treatment block, aeration block and sludge treatment block, was used.

1.2 Centralized supervision and control

1.2.1 Centralized supervision and control of the whole sewage treatment plant

In the Hokubu S.T.P. which was put into operation in 1968, a centralized supervision and control system taking the whole plant as its subject, was applied for the first time.

The outline of this system was as follows:

a) Monitoring was conducted through a graphical monitoring panel, to enhance the monitoring function. The processing system was graphically indicated on the front of the panel, and indicating meters (flow meter, opening meter, voltmeter, ammeter, etc.) and indicating lamps were arranged at key points in the system. In addition, an ITV unit was installed for monitoring the major site locations (i.e. the grit chamber, power station and sludge dewatering room).

b) Controls were, in principle, automatic; and in case that the equipment failed to function automatically, it would be automatically switched to the spare one.

A control desk was installed, separately from the monitoring panel, and ON–OFF switches, provided on the desk for the major components of the equipment (i.e. the influent gates, screens, main pumps and blowers), also made it possible to operate the equipment manually from the central operation room.

c) The measured values were indicated on the graphical monitoring panel, and, in addition, the data necessary for operation and maintenance were automatically recorded in the operation and maintenance diary, in a definite format at definite time intervals by a data logger.

The above-mentioned system made a one-man control system available, and a reduction of operators was visualized, compared with the conventional supervision and control system by blocks. A reduction of 16 operators was made, compared with the conventional supervision and control system by three blocks.

1.2.2 Remote supervision and control of relaying pumping station

Remote supervision and control was first introduced, in the newly installed relaying pumping station for the Chubu S.T.P. in 1970. (The distance between the pumping station and the plant is 7.5 km.) This system is dependent upon the Cyclic Digital type remote supervision and control equipment, using an exclusive cable. The application of this system reduced the number of staff necessary to operate and maintain the pumping station to only one for daily work of cleaning around the grit chamber and unmanned at night, while periodic inspection and maintenance work was carried out by two members of the staff desptached from the sewage treatment plant. (A conventional pumping station needed nine staff members for operation and maintenance.)

Also at that time, based upon experience in the Hokubu S.T.P., a data logger was introduced into the Chubu S.T.P. to prepare operation and maintenance diaries, for both the sewage treatment plant and relaying pumping station.

1.2.3 Application of electronic computer

"What can be replaced by the computer, among the works conventionally conducted by man, are to be conducted by the computer, and what can be conducted only by the computer are to be practically conducted by the computer." This is the purpose of the introduction of the computer. The computer was first used in the Totsuka 2nd S.T.P. and the Kohoku S.T.P., both of which were put into operation in 1972, with the purpose of the stabilization of treatment and rationalization of the operation and maintenance. The outline of the supervision and control system in the Kohoku S.T.P. is as follows:

a) The processes in the sewage treatment plant and relaying pumping station are continuously supervised by the computer, and occasionally by the operator.

b) A Mosaic type graphical monitoring panel was applied for an easy expansion and modification in the future.

c) Recording of process failures, and preparation of daily and monthly reports on operation and maintenance, are conducted by the computer.

d) The principle of operation in the process is automatic control, while the speed and number of units of the main pumps are directly controlled by the computer.

e) The relaying pumping station is, in principle, automatically operated at site, but occasional supervision and manual operations from the sewage treatment plant are possible through the remote supervision and control equipment. (The distance between the pumping station and the sewage treatment plant is 1.5 km.)

As the result of application of the above-mentioned system, the number of staff necessary to operate and maintain the process were reduced to 31, a decrease of 16, compared with the past.

1.2.4 Programable logic controller and micro-controller

The programable logic controller and micro-controller were first installed in the City in 1972, and made it possible to easily cope with modification and expansion, in the contents of control, in the future.

a) The programable logic controller was adopted for a blanket program control of feed, transfer, and withdrawal, of the sludge of the nine digestion tanks in the Nanbu S.T.P.
While the modification of the program is difficult in a conventional sequential circuit consisting of wired relay, it is easy in a programable logic controller which can use any other logic circuit when necessary.

b) The micro-controller was adopted for smooth control of the speed and number of units of the main pumps in the Isogo pumping station. While the flow rate control through conventional analog instruments was complicated, the adoption of the microcontroller having a microcomputer visualized easy access to optimum control, through arithmetic operation as well as an easy response to modification in the set flow rate and modification of or additions to the control program.

1.2.5 Improvement of the existing facilities, etc.

a) Since the management of the sludge treatment facilities in the Chubu S.T.P. and the Nanbu S.T.P. had been merged with the Sewage Works Bureau in 1965, the computer system was introduced into the Nanbu S.T.P. in 1974 and into the Chubu S.T.P. in 1975, with centralized supervision and control system adopted.
At that time, a simplified remote supervision and control equipment (simple super) was installed to operate remote monitoring and control of the sludge treatment facilities, the data logger already installed in the Chubu S.T.P. was removed, and its functions shifted to the computer.

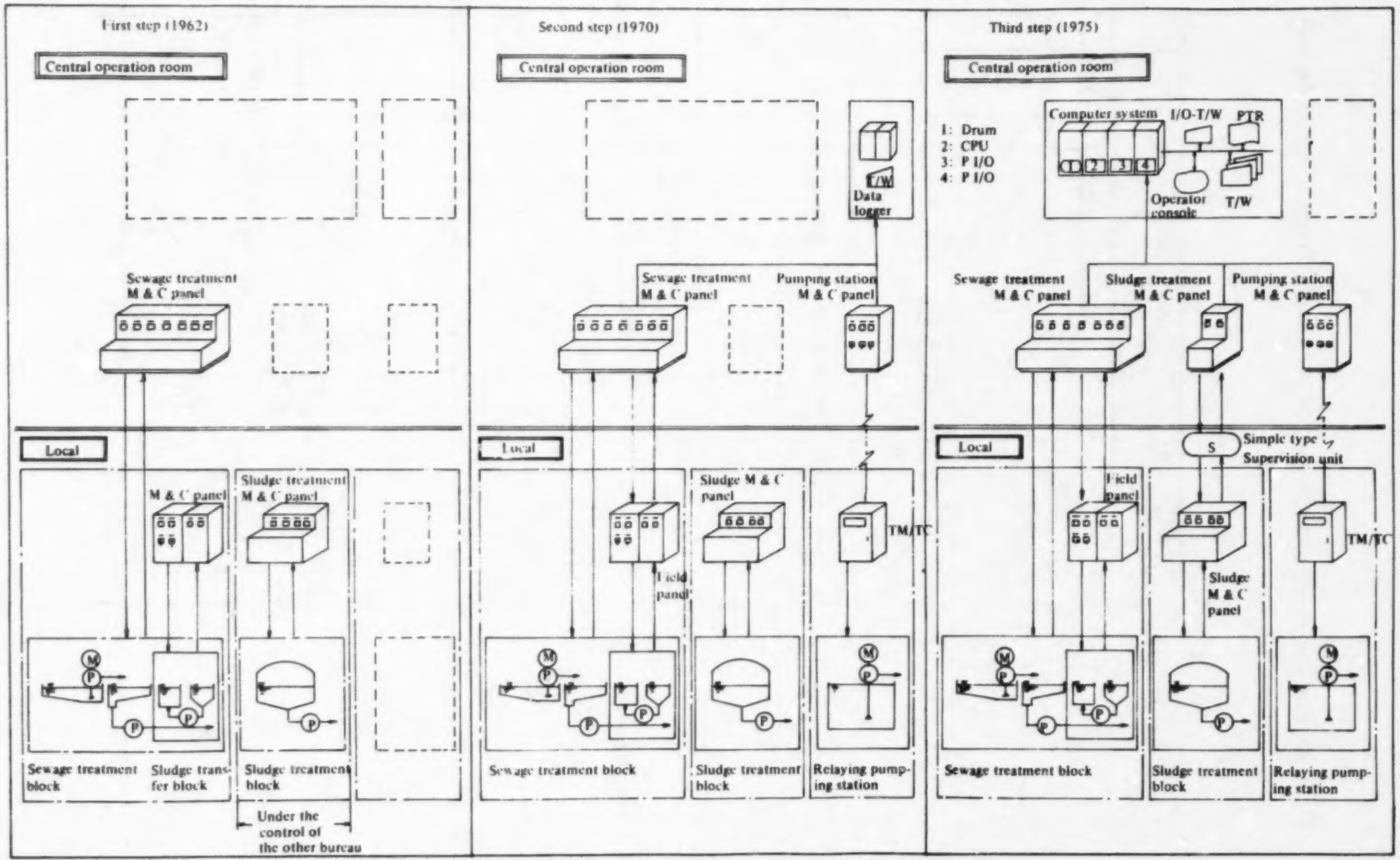
b) The micro-controller was incorporated into the control of the sludge treatment facilities of the Kohoku S.T.P., to operate liquid level control of the digestion tanks, chemical dosage control, control of the number of units of the press filters, etc., while a programable logic controller was incorporated in the operation control circuit. The combination of these two types of equipment was aimed at an optimum process control, the possible addition of hardware, and modification of the software.

c) In 1977, the computer system was applied in the Hokubu S.T.P. and a remote supervision and control system was used in the four relaying pumping stations, making it possible to control the volume of the sewage inlet flow from each relaying pumping stations into the sewage treatment plant.

d) In the Midori S.T.P., which was put into operation in 1977, a micro-controller was adopted for power control, resulting in an automatic switch from buying power to power station and vice versa, as with the automatic sequential ON–OFF of high voltage loads.

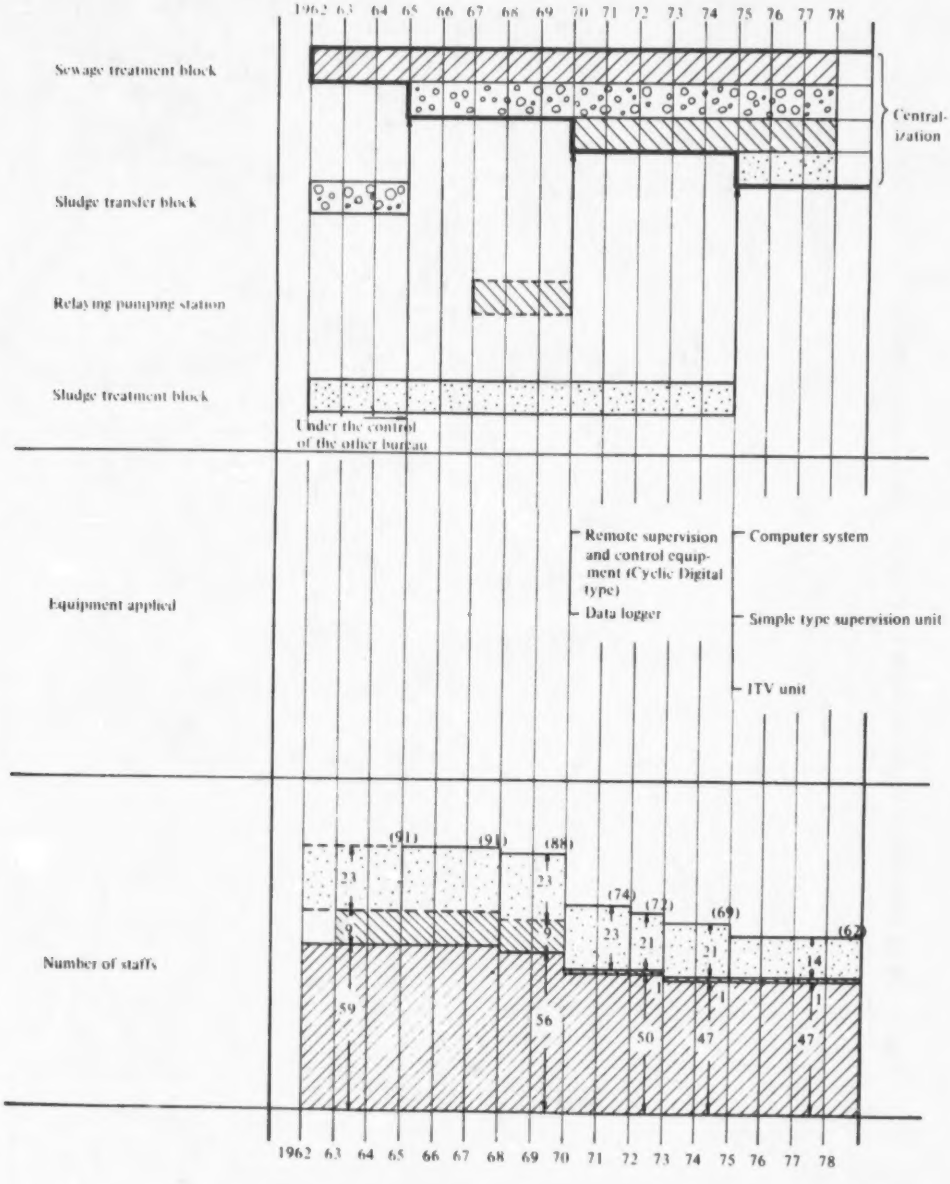
As stated above, during the period from 1962 to 1977 there was a transition from centralized supervision and control by blocks, to that in the aggregate, including the relaying pumping stations. Taking the Chubu S.T.P. as an example, the situation is as shown in Fig. 2 and Fig. 3.

Fig. 2 Centralization of supervision and control in the Chubu Sewage Treatment Plant (1)



TM/TC = (Remote Telemetry and Telecontrol unit)   M & C = (Monitoring Control)

Fig. 3 Centralization of supervision and control in the Chubu Sewage Treatment Plant (2)



2. Centralized supervision and distributed control system

The supervision and control system of the Kanagawa S.T.P., which has put into operation in 1978, has a newly developed basic ideas, though it is an extension of the automation. Briefly, "both supervision and control at center" has proceeded to "supervision at center, control at site".

2.1 Basic ideas for supervision and control in the Kanagawa Sewage Treatment Plant

The reason why the supervision and control system in the Kanagawa S.T.P. has been looked at other than as an extension of the traditional automation is as follows:

a) This plant has the largest treatment capacity in the City.

b) The equipment for automation and electrical data processing has been progressively developed, and the actual results have accumulated.

c) Further progress in man power saving has been required.

There has been serious discussion on what should be the optimum supervision and control system for large plants, resulting in the following policy:

a) The operation of the sewage treatment plant and relaying pumping stations should be conducted by a small number of operators (ordinarily one or two) through further promotion of automation and the qualitative enhancement of control.

b) Measures should be taken to prevent failure of the control at center, or in any part from having an influence on the whole.

c) The control system should be such that the optimum control mode can be selected in response to the process conditions, and modification can readily be made to cope with possible technological developments.

d) The technology for quality control should be progressively introduced.

Based upon the above-mentioned ideas, the monitoring of the processes in the sewage treatment plant and relaying pumping stations is concentrated into, and conducted by the computer installed in the central operation room, while the controls are multiplely conducted by giving a control function to each field block.

The advantages of this system are as follows:

a) The failure of the central computer does not lead to a stoppage of the whole plant, as control is continued in the field blocks.

b) As the controls are divided into each field block, it is possible to build a control program that matches best with the characteristics of each block.

c) When modification or addition is required in the control program, it can be made in each program, having no influence on the other blocks.

d) It is convenient for inspection and maintenance.

(The number of staff for management of the Kanagawa S.T.P. and two relaying pumping stations is 43.)

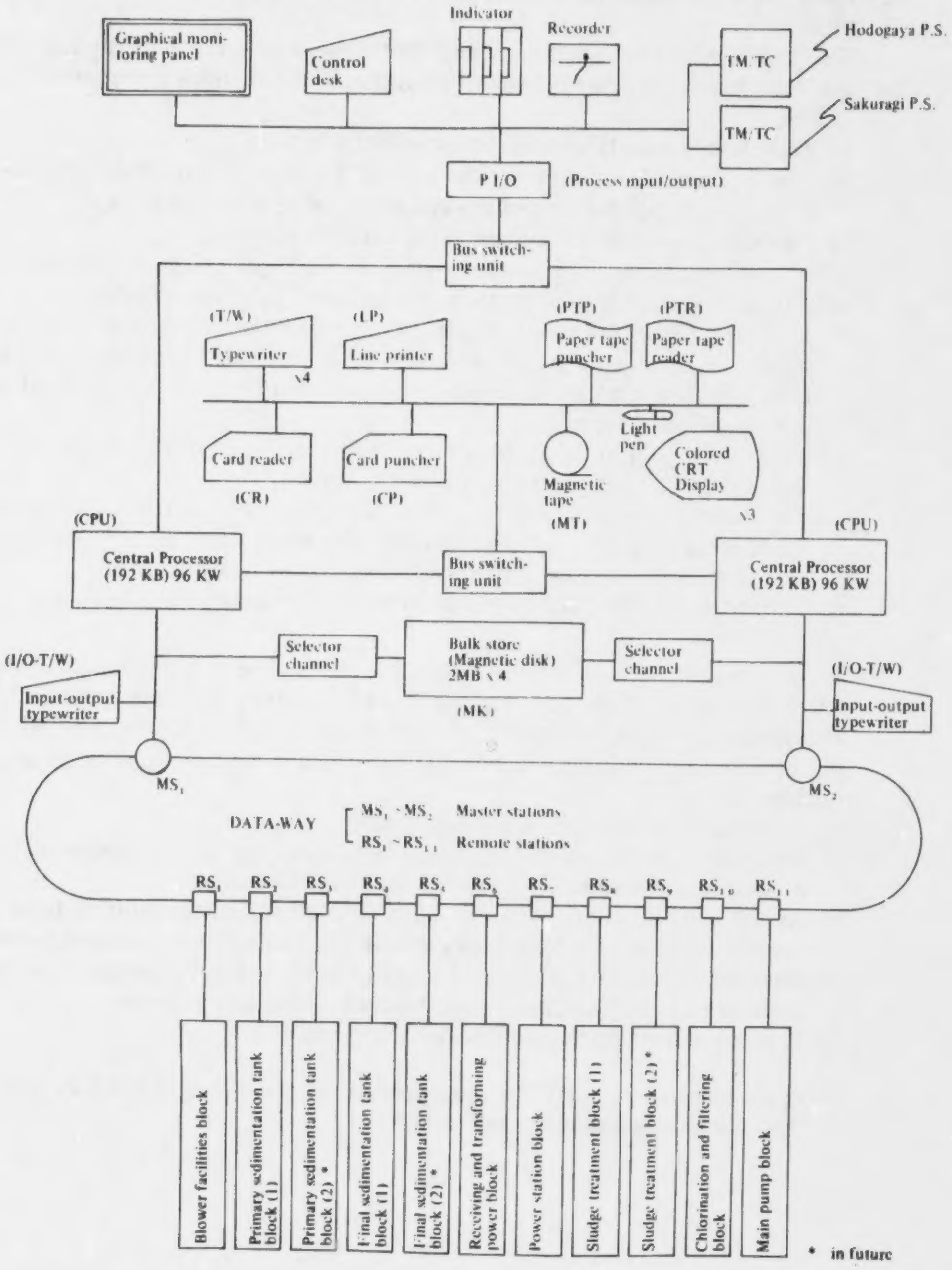
2.2 Outline of supervision and control system

The new system, in the Kanagawa S.T.P., is as shown in Fig. 4.

2.2.1 Central monitoring system

The central computer system has the following functions:

Fig. 4 Supervision and control system in the Kanagawa Sewage Treatment Plant



a) All process data for the sewage treatment plant and two relaying pumping stations are input to the computer, and the process conditions are continuously monitored.
b) In order to efficiently conduct the supervision and operation of the process, a greatly consolidated graphic monitoring panel of mosaic type, showing indicating lamps and system operations at a glance, is installed.
c) Process values and operating conditions of the equipment are indicated by colored CRT display equipments, and the instruction for selection of the control mode of the micro-controller of each field control block, modification of set values, setting of operation sequence of the equipment, etc. can be issued after viewing the CRT picture.
d) Process failure is automatically recorded, and daily, monthly and yearly reports on the operation are automatically prepared. The automatically measured data for the quality are not only displayed on the CRT picture, but also automatically listed on the daily, monthly and yearly reports, including manually analized quality data.

In addition, in order to enhance the reliability of the supervision and control function, the central computer is consisted of dual-duplex system.

2.2.2 Control system in field blocks

The constitution of the control system in each field block is as shown in Fig. 5.

a) The control in each field block is perfomed in the minor loop by the micro-controller, programable logic controller, digital instruments, etc. For the ON–OFF instructions, set values etc., the control outputs are issued from the processing unit of the central computer, through the DATA-WAY described later. Similarly, the data from each field control block are sent, through the DATA-WAY, to the data processing unit of the central computer, and annunciation and memory are conducted.

Fig. 5 Block diagram of control system in field block

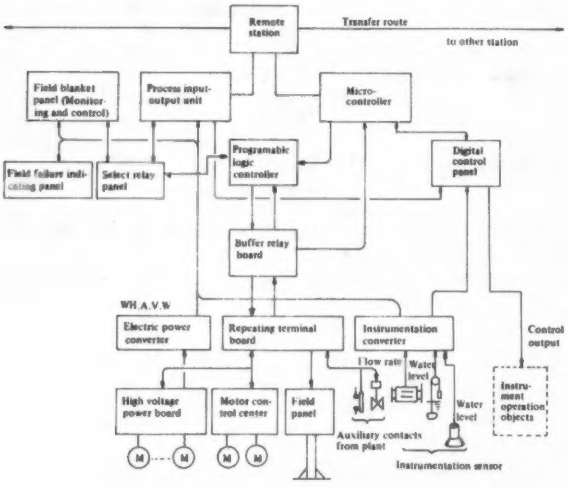


Table 1 Field Block Items in Kanagawa Sewage Treatment Plant

Field Block		Control Item
1. Receiving and transforming power block		1. Interruption and recovery control 2. Power supervision and control Received power Generated power 3. Power-factor control
2. Power station block		1. Automatic starting control 2. Generating energy control Simulated load control Number of units control 3. Continuous priming control
3. Main pump block	Main pump	1. Optimum level control (Velocity control and Number of units control) 2. Optimum flow control (Velocity control and Number of units control)
	Gate	Automatic opening and closing control by sewage inflow volume
	Screen	1. Screen control by level difference 2. Time control and interlocking control
4. Primary sedimentation tank block (1)		Sludge withdrawal control (Constant volume, proportional to sewage inflow and primary sedimentation tank sludge level control)
5. Aeration tank and final sedimentation tank block (2)	Aeration	Air volume control (Constant volume, proportional to sewage inflow, or set DO control)
	Return sludge pump	Flow rate control (Constant volume, proportional to sewage inflow, or final sedimentation tank sludge level control)
	Excess sludge pump	1. Intermittent withdrawal (preset control and timer control) 2. Continuous withdrawal (set MLSS control)
6. Blower facilities block		1. Suction air volume control (Regulating valve control) 2. Number of units control
7. Sludge treatment block		1. Sludge volume control 2. Chemical dosing control 3. Dewaterer optimum process control
8. Chlorination and filtering block		1. Constant dosing volume control 2. Constant dosing rate control
Primary sedimentation tank block (2), an aeration tank and final sedimentation tank block (2) and sludge treatment block (2) may be added in the future.		

b) The field blanket panel, installed in each field block, shares the same functions as the operating function of the central desk and the condition-indicating function of the colored CRT display equipment, enabling the operation of the equipment at site, if necessary. The control items of each field block are given in Table 1.

2.2.3 DATA-WAY (Multi-data transmission) system

This is a system where the master station is located in the central computer and the remote stations are located in each field control block, and a great deal of data necessary for monitoring and control can be voluntarily transferred between the stations at high speed. Since both stations are connected by a pair of cables, this system is superior to the conventional control cable system in the following points:

a) No limit is required in the type and quantity of data to be transferred.
b) It is easy to cope with a possible increase in the data to be transferred.
c) A shorter period is required for the design and construction of the cable.
d) The cost for wiring is greatly reduced.
e) Inspection and maintenance are easier.
f) Remedy of the cable failure is facilitated by doubling the transfer cable.

Conclusion

In the City of Yokohama, the automatic control and recording of various processes, and the remote supervision and control of the relaying pumping stations, have been promoted for the past 16 years, with the aim of a centralized supervision and control in operation and maintenance of the sewage treatment plants.

This has been supported by the introduction of electronic computers and the development of related supervision and control equipment. As a result, at present, normal monitoring, control and recording are all performed by the equipment, the operators being required to monitor only occasionally.

If the subject of monitoring and control becomes more plentiful and more complicated, the centralized supervision and distributed control system, where supervising function and control function are separated is taken to enhance the reliability, flexibility, and economy of the system. This greatly dues to the advent of the microcomputer which is suitable in capacity and profitable in the economy of the number of loops used, from a few to several dozen, and the DATA-WAY, which makes a high-speed transfer of a great deal of data possible. Thanks to this advanced technology, a reasonable general management of the whole process has become available through an automatic control of each block, that composes the whole system, and a smooth transfer of the data in the whole system.

Up to the present, however, the main subject of supervision and control is quantitative control, and to obtain the optimum control, technology in the field of qualitative control must be developed. As a part of such development, the City is promoting field tests for some of quality sensors, and has already obtained a practical use for some of them. We plan to steadily accumulate the actual results, as our theme for the future.

CHAPTER 6
MISSION AND ACTIVITIES OF JAPAN SEWAGE WORKS AGENCY AT PRESENT AND FUTURE

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1. MISSION OF JAPAN SEWAGE WORKS AGENCY

1.1 SEWERAGE SYSTEMS BUILT BEFORE 1950s

Before taking up the main subject – the background to the establishment of the JSWA, it would be necessary to briefly review the history of construction of sewerage systems in Japan. Construction of modern sewerage systems for cities began with the enactment of the Sewerage Act in 1900. According to this Act, construction and management of sewerage systems were entrusted with each city, but when a city wanted to provide its sewerage system, an approval of the plan by the Minister in charge was necessary. In spite of the enactment of the Sewerage Act, sewerage systems were not actually built before 1945 except in a limited number of large cities. As of 1945, only the following six cities had sewage treatment plants in operation.

After 1945 and during the 1950s, sewage works did not make a development so much all over Japan, and the rate of provision with sewerage systems remained at a very low level. This situation was due mainly to the following reasons:

(1) The lavatory at each household was a dipping-up type, and night soil was carried to a farm land to be used as manure for crops.

(2) Necessity of building sewerage systems was not well recognized.

(3) Municipalities which were responsible to employ their sewerage systems construction were in difficult positions to obtain appropriate funds for the construction of sewerage systems.

(4) In those days, the government's top priority policy was to form the basis of industries; measures and facilities for environmental protection such as sewerage systems were left over.

City	Name of Sewage Treatment Plant	Treatment System	Start of Operation
Tokyo	Mikawashima	Trickling Filter Process	1923
Nagoya	Horidome	Activated Sludge Process	1928
Nagoya	Atsuta	Activated Sludge Process	1930
Nagoya	Tsuyuhashi	Activated Sludge Process	1932
Kyoto	Kisshoin	Activated Sludge Process	1934
Toyohashi	Noda	Activated Sludge Process	1935
Gifu	Chubu	Activated Sludge Process	1937
Tokyo	Mikawashima	Activated Sludge Process	1937
Tokyo	Shibaura	Activated Sludge Process	1937
Kyoto	Toba	Activated Sludge Process	1938
Osaka	Tsumori	Activated Sludge Process	1940
Osaka	Ebie	Activated Sludge Process	1940

1.2 ENVIRONMENTAL POLLUTION AFTER 1960 AND ITS COUNTERMEASURES

Environmental pollution rapidly developed in the 1960s. A number of reasons can be cited for this situation, but the first reason is that Japan attained a remarkable economic growth in the 1960s, with a shift of its industrial structure from primary industries to secondary and tertiary industries. Also there was a population move from agricultural and fishing villages to cities. As seen in Table 1, in 1960, the ratio of urban population (population in densely populated districts) to the total population in Japan was 43.3%, but in 1975, this ratio increased to 57%. The total area of urban districts in Japan was 3,865 km^2 in 1960, but this figure also increased to 8,275 km^2 in 1975, ushering in the age of urbanization and industrialization. As is clear from Table 2, the rate of increase of the urban population in Japan is higher than that of other major countries of the world.

The second reason is that the economic growth rate, especially that of the industrial sector, was exceptionally high as shown in Table 3.

Table-1 Periodical Change of City (Urban District) Population

	Whole Country		Cities (Urban Districts)			Other Districts		
	Total Population (10 thousand A)	Total Area (km^2)	Population (10 thousand) B	Area (km^2)	B/A (%)	Population (10 thousand) C	Area (km^2)	C/A (%)
1960	9,430	374,773	4,083	3,865	43.3	5,347	370,908	56.7
1965	9,920	375,070	4,726	4,605	47.6	5,194	370,465	52.4
1970	10,466	377,535	5,599	6,444	53.5	4,867	371,091	46.5
1975	11,194	377,535	6,382	8,275	57.0	4,812	369,260	43.0

Table-2* Ratio of City Population (1970) and Annual Rate of Increase (1960 ~ 1970)

	Ratio of City Population to Total Population in 1970 (%)	Annual Rate of Increase for Years 1960 ~ 1970 (%)
Japan	56.3	4.0
U.S.A.	58.3	2.8
Britain	71.7	0.5
France	42.6	3.5
Italy	29.4	2.8
Sweden	32.7	3.5
Netherlands	45.2	3.1
OECD	49.3	2.7

*Environmental Policies in Japan, OECD, Paris 1977

Table-3* Growth Rates of Japan and Major Member Countries of OECD (Annual Rate, 1960 ~ 1970, %)

·	GNP	Industrial Output	Energy Consumption	Number of Automobiles in Use
Japan	10.8	14.8	11.6	25.3
U.S.A.	4.2	4.8	4.5	3.7
Britain	2.7	2.8	2.3	6.6
France	5.6	5.9	5.3	8.2
Italy	5.5	7.0	8.9	24.2
Sweden	4.6	6.1	5.0	6.4
Netherlands	5.3	7.3	8.4	15.7
OECD	5.0	5.9	3.0	6.2

*Environmental Policies in Japan, OECD, Paris 1977

What played inportant roles in the economic growth process during 1960s were such pollution-causing industries as the steel making, power generation, cement production, paper and pulp manufacturing, foodstuff processing and chemical synthesizing, all these marking particularly high growth rates. During this period, there were several incidents in which health of residents in relevant areas was greatly impaired due to water contamination by local industries.

For example, in 1959, residents in Minamata City, Kumamoto Prefecture, contracted a disease after they ate fish contaminated by mercury which was discharged into Minamata Bay by a nearby chemical factory. Since then, this disease has been named "Minamata Disease". In 1965, similar cases to those of Minamata Disease were found in the area along the Aganogawa River in Niigata Prefecture. This was also proved as mercury contamination due to discharge of waste water from a local chemical factory. In 1959, a cadmium contamination case called "Itai-itai Disease", was found in the basin of the Jintsu River, Toyama Prefecture. The cause of this social problem was the waste water from a metal mining company in the area.

The third reason is that the production and consumption activities in Japan were concentrated in limited areas. Japan is an island country with a population of more than 100 million, and the population density is very high. A greater part of the land is mountainous, and the habitable area with a ground surface gradient of less than 10% is only a quarter of the total land area.

All the production and consumption activities are conducted within this habitable area, therefore, the activities per km^2 of the habitable land area – the pollution load – are greater than in any other country of the world as shown in Table 4.

The fourth reason is that historically very little public investment has been made in Japan for the stock of the social overhead capital. In many cases, the social

Table-4* **Major Economic Activities per km² of Habitable Land Area**

	GNP (Million U.S. Dollars) 1975	Industrial Output (Million U.S. Dollars) 1974	Energy (1,000 tons in Terms of Petroleum) 1974	Number of Automobiles Owned 1974
Japan	6.05	2.04	4.12	331
U.S.A.	0.32	0.09	0.36	27
Britain	1.04	0.26	1.00	80
France	0.87	0.25	0.47	47
Italy	0.81	0.24	0.66	74
Sweden	1.67	0.44	1.09	69
Netherlands	3.10	0.83	2.38	146
OECD	0.31		0.27	21

*Environmental Policies in Japan, OECD, Paris 1977

Table-5 **Comparison between Rate of Provision with Water Supply Facilities and That of Sewerage Systems**

	Total Population A (10,000 people)	Population Supplied B (10,000 people)	Population Sewered C (10,000 people)	B/A (%)	C/A (%)
1960	9,342	4,991	519	53.4	6.1
1965	9,827	6,824	816	69.4	8.3
1970	10,372	8,375	1,630	80.8	15.7
1975	11,194	9,839	2,551	87.8	22.8

Table-6 **Periodical Change in GNP and Investment in Sewerage System Construction**

	GNP A (100 million Yen)	Investment in Sewerage System Construction B (100 million Yen)	Ratio (B/A) %
1960	162,070	203	0.12
1965	328,137	705	0.21
1970	730,461	1,893	0.26
1975	1,495,500	6,975	0.46
1977	1,860,140	11,617	0.62

overhead capital contributes to reducing the pollution load, but in Japan, the social overhead capital was accumulated slower than in other countries. Moreover, the social overhead capital is not well balanced in its contents, which brings about environmental pollution. For example, Table 5 shows a comparison between the rate of provision with water supply facilities and that of sewerage systems. This is an indication that, as a whole country, water supplied and used is being discharged without any control and without any proper treatment.

The fifth reason for the environmental pollution is that since 1945 Japan has regarded industrial development as a matter of prime importance and the government policy for the industrial development has resulted in relative negligence of the quality of the environment. Since 1970, however, there has been a slight change in its policy, the quality of the environment being given serious consideration. Many of the countermeasures taken were rather patchy, and no satisfactory results have been obtained so far. Recently, however, improvement in the environmental quality has been somewhat felt.

Let's look at the investment made in sewerage systems as a water pollution control measure. As is clear from Table 6, the ratio of funds invested in sewerage systems to GNP was 0.12% in 1960, but it increased to 0.62% in 1977. If we look at the public investment plan included in the government's economic plan, we see that the investment in the sewerage systems gradually increases, as seen in Table 7, being given a sufficient consideration along with road construction, housing development and flood control.

Table-7 Social Capital Investment and 5-year Construction Plan for Sewerage System in Government Economic Plan

(Unit: 100 million yen)

Economic Plan / Public Works	Economic Plan for Late 1970s (1976 ~1980)		Basic Economic and Social Plan (1973 ~ 1977)		New Economic and Social Development Plan (1970 ~ 1975)		Economic and Social Development Plan (1967 ~1971)		Economic Plan for Mid-1960s (1964 ~ 1968)	
	Investment	Share (%)	Investment	Share (%)	Investment	Share (%)	Investment	Share (%)	Investment	Share (%)
Road Construction	195,000	19.5	190,000	21.1	117,000	21.3	61,500	22.4	41,000	23.0
Housing	65,000	6.5	60,800	6.8	39,000	7.1	17,100	6.2	11,200	6.3
Sewerage Systems	71,000	7.1	56,500	6.3	23,000	4.2	9,300	3.4	5,792	3.3
City Parks	15,400	1.5	13,000	1.4	4,300	0,8	2,070	0.7	805	0.4
Food Control	55,000	5.5	47,000	5.2	29,000	5.3	16,100	5.9	9,000	5.1
Total	401,400	40.1	367,300	40.8	212,300	38.6	106,070	38.6	67,797	38.1
Grand Total	1,000,000	100.0	900,000	100.0	550,000	100.0	275,000	100.0	178,000	100.0
5-year Construction Plan for Sewerage	fourth				third		second		first	
Planned Period	1976 ~ 1980				1971 ~ 1975		1967 ~ 1971		1963 ~ 1967	
Investment	75,000				26,000		9,300		4,400	

1.3 HISTORY OF WATER POLLUTION CONTROL AND THE SEWERAGE ACT IN JAPAN

Rapid improvement was seen in the 1970's for the protection of the environment, particularly with the sewerage systems of the country. It appeared against the backdrop of deteriorating living environment of the 1950's and 1960's.

The followings are some of the major developments in chronological order:

1958	The Sewerage Act (April 24)
	A Law of Quality Control of Public Water (December 25)
	A Law of Industrial Waste Water (December 25)
1959	The Sewerage Section established within the Public Works Research Institute, Construction Ministry (April 1)
1963	The Act For The Improvement Of The Living Environment And Facilities (December 24)
	The First Five-Year Sewerage Construction Plan (with the estimated budget of 440 billion Yen)
1967	The Act Of Sewerage Improvement (June 21)
	The Basic Law For Environmental Pollution Control (July 21)
1969	The Second Five-Year Sewerage Construction Plan (with the total budget of 960 billion Yen; February 21)
1970	A Class of the Environmental Water Quality Standard decided by the Government (April 1)
	The Water Pollution Control Act (December 25)
	Partial Revision of the Sewerage Act (December 25)
1971	The Establishment of the Department of Sewerage & Sewage Purification within the Ministry of Construction (April 1)
	The Establishment of the Environment Agency (May 31)
	The recommendations by the Central Committee on City Planning, concerning "Ways and means to execute the promotion of sewerage construction" (August 16)
	The Third Five-Year Sewerage Construction Plan (at the total expense of 2.6 trillion Yen; August 27)
	The First US/Japan Conference On Sewage Treatment Technology (October 25 ~ 28), Tokyo, Japan
1972	The Sewage Construction Project Center Act (May 29)
	The Second US/Japan Conference On Sewage Treatment Technology (November 27~December 8), Cincinnati, Ohio and Washington, D.C., U.S.A.
1973	The Basic Economic And Social Plan (1973 ~ 1977) approved by the Government (February 13)
	The Seto Inland Sea Conservation Law (October 2)
1974	The Third US/Japan Conference On Sewage Treatment Technology (February 12 ~ 16), Tokyo, Japan
1975	The Japan Sewage Works Agency Act (February 21)
	The Fourth US/Japan Conference On Sewage Treatment Technology (From October 22 ~ 30), Cincinnati, Ohio and Washington, D.C., U.S.A.

1.4 THE BACKGROUND FOR THE ESTABLISHMENT OF THE JAPAN SEWAGE WORKS AGENCY

In the 1970's, various measures were introduced by the Government to construct and improve the sewerage systems as part of its efforts to control the quality of water. Particularly, with the start of the third Five-Year Sewerage Construction Plan in 1971 at the expense of 2.6 trillion Yen, which is a little less than three times the amount of the second Five-Year Plan, sewerage works increased tremendously, and as a result, we saw a lack of enough engineers to carry out the work smoothly.

On April 21, 1970, the Government approved a Class of the Environmental Quality Standard. And consequently, the number of rivers increased, whose water qualities must meet the government standard. And in order to meet the national standard to control the water quality of rivers in Japan, the Government had to construct sewerage systems of local municipalities. As shown in Table 8, the governmental emphasis on sewerage construction has been shifted from big cities to smaller-sized local cities and towns, and this trend will become more so in the future.

It is very important of employ enough well-trained engineers for sewerage construction. And such engineers are usually concentrated in big cities in Japan. This is because, in large cities, a fairly large amount of work is available continuously at higher pay than in smaller cities. On the other hand, in smaller cities and prefectural governments, there are not enough engineers both of them have not conducted sewage works so far and have not employed engineers in charge of these works. Under these circumstances it would be difficult for them to carry out their sewerage projects particularly big-scale sewerage construction projects. They are not well experienced and it is difficult for such local municipalities to employ specialized engineers, such as sanitary, electric, mechanical engineers, chemists and biologists in charge of planning, constructing, and maintaining sewage treatment plants. In addition, each local municipality is an independent organization, personnel exchanges are virtually impossible between big cities and smaller ones. This is due mainly to Japan's life-time employment system, under which a person is hired by an organization for life and he is expected to stay there for life.

Private consulting firms in the field of sewage works in Japan have come into existence only recently, and generally there are few which have well-experienced, well-qualified engineers.

Under these circumstances, the Minister of Construction asked the Central Advisory Committee on City Planning to submit its recommendations on ways to execute the promotion of sewerage construction. And on August 16, 1971, the Committee came up with the recommendations that the Government and local governments must take fundamental measures as soon as possible to provide the system under which engineers are to be pooled and sent to various organizations as the need arises. Along the line of the recommendations, the Ministry of Construction asked the Education Ministry to put more emphasis in providing courses on sanitary engineering in technical colleges and universities. The Ministry of Construction also began preparation, with cooperation of local governments, to establish a sewerage works organization for the purpose of pooling enough engineers to be dispatched where they are needed, as well as advancing the construction expenses

Table-8 Sewerage Investment In Big And Smaller-sized Cities

Year	10 big cities		Smaller-sized cities		Total	%	
	Amount Invested (billion Yen) "A"	No. of cities	Amount Invested (billion Yen) "B"	No. of cities	Amount Invested (billion Yen) "C"	A/C	B/C
1960	15.0	10	5.4	138	20.4	73.6	26.4
1965	48.9	10	21.6	162	70.5	69.4	30.6
1970	112.4	10	77.0	239	189.4	59.3	40.7
1975	361.4	10	336.2	476	697.6	51.8	48.2

for the needy local municipalities. And the Sewage Construction Project Center (SCPC) was set up on November 1, 1972.

Among its many functions, this SCPC's main job is to give various technical assistance to local governments, including the assessment of sewerage construction projects, the planning and actual construction works under commission with their money, as well as other assistance, such as training of engineers and research and technology development and in making it into practical use in cooperation with local governments each giving 50% of the total cost. In short, the SCPC has been working to help local government bodies to promote sewerage works in Japan. However, since then, the application of the water quality standard by the Government was extended to rivers, lakes, bays and coastal waters throughout Japan, and as a result, the construction of sewerage systems for environmental protection became to be beyond the boundaries of one local government. It came to be recognized, instead, as the national issue to be immediately dealt with in order to conserve the nation's water resources as a whole. Consequently, the main function of the SCPC was being shifted from giving assistance to local governments to taking over their operations to construct sewerage systems by the SCPC.

With the changes in the nature of the SCPC during the first three years after its establishment, the Japan Sewage Works Agency Act was enacted on June 19, 1975, which virtually absorbed the SCPC into a new body, the Japan Sewage Works Agency by enlarging the functions and organizational structure of the former.

1.5 LEGAL ASPECT OF THE JAPAN SEWAGE WORKS AGENCY

The JSWA is a public corporation which has a special status as stipulated by a special law. Since its operation involves the sewage works which local governments ought to do, the JSWA must work for the benefit of all these local governments. Therefore, the establishment of the JSWA is to be initiated by the representatives of local governments, and the cost of establishing the organ and some operational expenses are to be borne equally by the central government and local governments, including the cost for training engineers and for research and technology development. Planning and construction of sewerage systems by the JSWA under commission with a local government is to be paid for by the local government concerned. However, since the function of the JSWA is not only to promote common interests among local municipalities by providing sewerage systems to improve the living environments of the people, but also to attain the national goal to control the quality of the public water and to secure water resources for the entire nation, the JSWA is considered to be the same as those established by the central government as far as the control and financial assistance of the Government is concerned.

1.6 OUTLINE OF THE FUNCTIONS OF THE JAPAN SEWAGE WORKS AGENCY

The functions of the JSWA are stipulated in the first clause in Article 26 of the Japan Sewage Works Agency Act. The following are some of the major functions of the organization:

(1) Construction of main sewerage facilities.

Under commission from a local government, the JSWA is to consturct sewage treatment plants, the trunk sewers to such plants, and the pumping stations.

(2) Designing, construction supervision, and operation and maintenance of the sewerage systems

Under commission from a local government, the JSWA is to plan sewerage systems, supervise their construction, operation and maintenance the sewage treatment plants and the pumping stations.

(3) Technical assistance for sewerage construction

By request of a local government, the JSWA is to give technical assistance for the assessment of the plans, construction, and the operation/maintenance of the plants.

(4) Tranining of engineers

The JSWA is to give training to national and local civil service personnel who are in charge of sewage works. Training of those from commercial companies by the JSWA is prohibited.

(5) Qualification tests

The JSWA is authorized to give qualification tests to those who wish to take them, including engineers of both national and local governments as well as those from private companies.

(6) Research and technology development

The JSWA is to conduct research and technology development in the field of sewerage, sewage treatment and pre-treatment, as well as on the practical application of such technology.

(7) Construction of the facilities, and giving technical assistance upon request of public corporations

Under commission from public corporations which are established by special laws, the JSWA is to plan and construct the main sewerage facilities, supervise the actual construction work, and give technical assistance on operation of the facilities.

1.7 FINANCING AND SUBSIDIES FOR THE JAPAN SEWAGE WORKS AGENCY

The JSWA receives capital fund and subsidies from the central and local governments. The capital fund covers the costs of land purchase and the construction of the administration building and the laboratory, as well as the deposit for renting the office building, housing for staff members, and others. In other words, the capital fund is used to acquire the fixed property for the operation of the JSWA.

On the other hand, the subsidies are to be spent on personal expenses of the staff in the planning, general affairs, accounting, training, and research and development divisions of the JSWA, as well as travel expenses and operational costs. Subsidies are also used to provide training facilities and testing equipments for training courses.

In principle, the central government and local governments each contribute half of the combined total amount of the capital fund and subsidies for the operation of the JSWA. With the half to be paid by the local governments, a group of prefectural governments and a group of local municipalities (each with more than 80,000 in population) contribute with the 2 to 1 ratio. In other words, the ratio of contribution to the finances of the JSWA is 50% by the central government, 25% by prefectural governments, and another 25% by local municipalities. This is based on the government recognition that the construction and operation of sewerage systems in controlling the quality of the public water should be carried out jointly by various governments at different political levels of the nation.

1.8 POLICIES IN CONDUCTING BUSINESS AT THE JAPAN SEWAGE WORKS AGENCY

(1) Conduct of business

Business is to be conducted according to the Japan Sewage Works Agency Act, its Enforcement Ordinance, and the detailed enforcement regulations. In practice, business is carried out according to the operational procedure which is approved by the Minister of Construction as stipulated in the regulations mentioned above, as well as according to regulations and official notices issued by the President of the JSWA.

The JSWA is also bound legally to submit its budget, plant of operation, and financial program each fiscal year to the Minister of Construction for approval. This means that the JSWA conducts its business within the regulations specified by law and others, as well as within its budget. Therefore, the JSWA's operations are secured both legally and financially. And the organization is expected to conduct its business most efficiently and properly as an organizational entity.

(2) Conclusion of an agreement

Since the JSWA operates under commission with a local government, the operational procedure manual directs the organ to conclude an agreement of commission with the local government concerned. the manual also specifically mentions the details to be agreed upon. This agreement of commission is the basis, on which other businesses are conducted thereafter. Howver, according to the regulations on conducting business under commission, an agreement of commission is to be concluded only after the approval of the national budget, the budgetary appropriation for the JSWA, and the business plans are thoroughly examined. This is because the operations of the JSWA are costly, such as the construction of a sewage treatment plant, and because the cost of such operation is to be financed by subsidies and other financial assistance of the national government.

After an agreement is reached on the nature of the commissioned operation, the date of completion, and other details, the JSWA makes a report on the agreement and sends a copy to its local government for approval. If no objection is voiced or no unexpected circumstances arise, the agreement goes into effect after it is signed by the representatives of both parties. The formal contract is to be made by the party which commissions the JSWA for work, bacause that party is primary responsible to do sewage works, and not the JSWA.

(3) Payment of expences

After the agreemtnt is signed by both parties, the JSWA has an obligation to complete the work under commission by a certain date specified in the contract. And the trusting party take responsibility to bear the cost of the work by its trustee, the JSWA. However, such rights and duties are different from those in ordinary contracts in that the fiduciary relations between the signatories rest at a much higher level than those in ordinary business contracts. And this difference is clearly seen in the payment of expences.

The trusting party pays necessary expenses in advance within the amount specified in the agreement, as the work advances, and upon completion of the commissioned work, the JSWA calculates accurately the amount to be borne by the trusting party. The payment of expenses is in fact a matter of settling accounts, which means that the amount of payment agreed on in the contract is essentially either the rough estimate or the ceiling amount of payment.

We are fully aware that the cost of a commissioned work is of primary importance particularly to the trusting party. And the JSWA goes by the detailed regulations on the payment of expenses, by which the contract amount of payment, or the rough estimate or the ceiling amount, can be rationally calculated that is acceptable to the trusting party.

(4) Delivery of completed work and settling accounts

Upon completion of the commissioned work, the JSWA hands the completed facilities or goods over to the trusting party and settles an account. With such facilities as sewage treatment plants, the maintenance and the trial run, and the problem of advance security are involved, and therefore, the JSWA has the special regulations, besides a contract, when delivering the final products to its trusting party. Other businesses are conducted according to what a contract stipulates.

1.9 THE ROLE OF THE JAPAN SEWAGE WORKS AGENCY AND THE PROCEDURES OF OPERATIONS

The JSWA can be regarded as an organ to achieve the central government's policy of controlling the quality of the public water mainly by constructing sewage treatment plants under commission with local governments. The role of the JSWA is stipulated in Article 1 of the Japan Sewage Works Agency Act, which says that "the JSWA constructs and maintains the main facilities of sewerage systems, gives technical assistance, trains sewage works engineers by request of local governments, as well as promoting research and technology development of sewerage systems and practical application of new technology development. And in so doing the JSWA is to promote the betterment of sewerage systems, imporve the living environment, and control the quality of the public water in Japan".

The procedures of the JSWA's operations are indicated in Table 9.

1.10 THE BUDGET AND THE ORGANIZATIONAL STRUCTURE OF THE JAPAN SEWAGE WORKS AGENCY

The budget of the JSWA for fiscal 1978 is shown in Table 10. The organizational structure of the JSWA is given in Table 11. The number of staff is 670, and the ratio of staff by job classification is shown in Table 12.

Table-9 Standard Procedures In Public Sewerage Construction Project

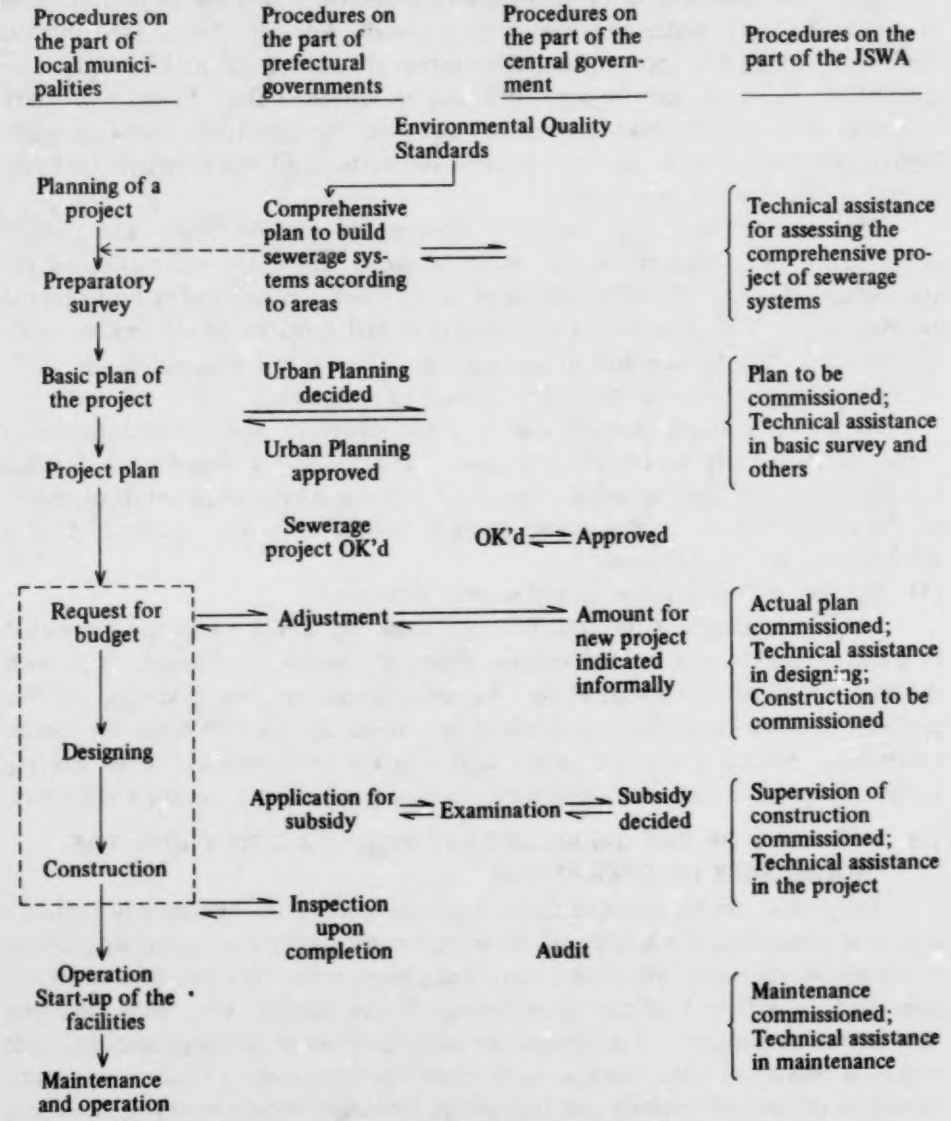
Procedures on the part of local municipalities
Procedures on the part of prefectural governments
Procedures on the part of the central government
Procedures on the part of the JSWA
Environmental Quality Standards
Planning of a project
Comprehensive plan to build sewerage systems according to areas
Technical assistance for assessing the comprehensive project of sewerage systems
Preparatory survey
Basic plan of the project
Urban Planning decided
Plan to be commissioned; Technical assistance in basic survey and others
Urban Planning approved
Project plan
Sewerage project OK'd
OK'd
Approved
Request for budget
Adjustment
Amount for new project indicated informally
Actual plan commissioned; Technical assistance in designing; Construction to be commissioned
Designing
Application for subsidy
Examination
Subsidy decided
Supervision of construction commissioned; Technical assistance in the project
Construction
Inspection upon completion
Audit
Operation Start-up of the facilities
Maintenance commissioned; Technical assistance in maintenance
Maintenance and operation

Table-10 The 1978 Budget of the JSWA

(Million Yen)

Categories	1978	1977	Rate of increase
Projects:			
Work under commission	98,953	79,197	1.25
Technical assistance	252	290	0.87
Training, qualification tests etc.	506	486	1.04
Management cost	17,112	14,759	1.16
Management cost	6,481	5,956	1.09
Refund	10,631	8,803	1.21
TOTAL	116,823	94,732	1.23
Budget:			
Capital fund	76	76	1.00
Central government	38	38	1.00
Local governments	38	38	1.00
Subsidies	904	896	1.01
Central government	452	448	1.01
Local governments	452	448	1.01
Loans	13,635	16,083	0.85
Revenue on commission	90,770	68,000	1.33
Revenue outside business	11,435	9,677	1.18
TOTAL	116,820	94,732	1.23

Table-11 The organizational Structure (1978)

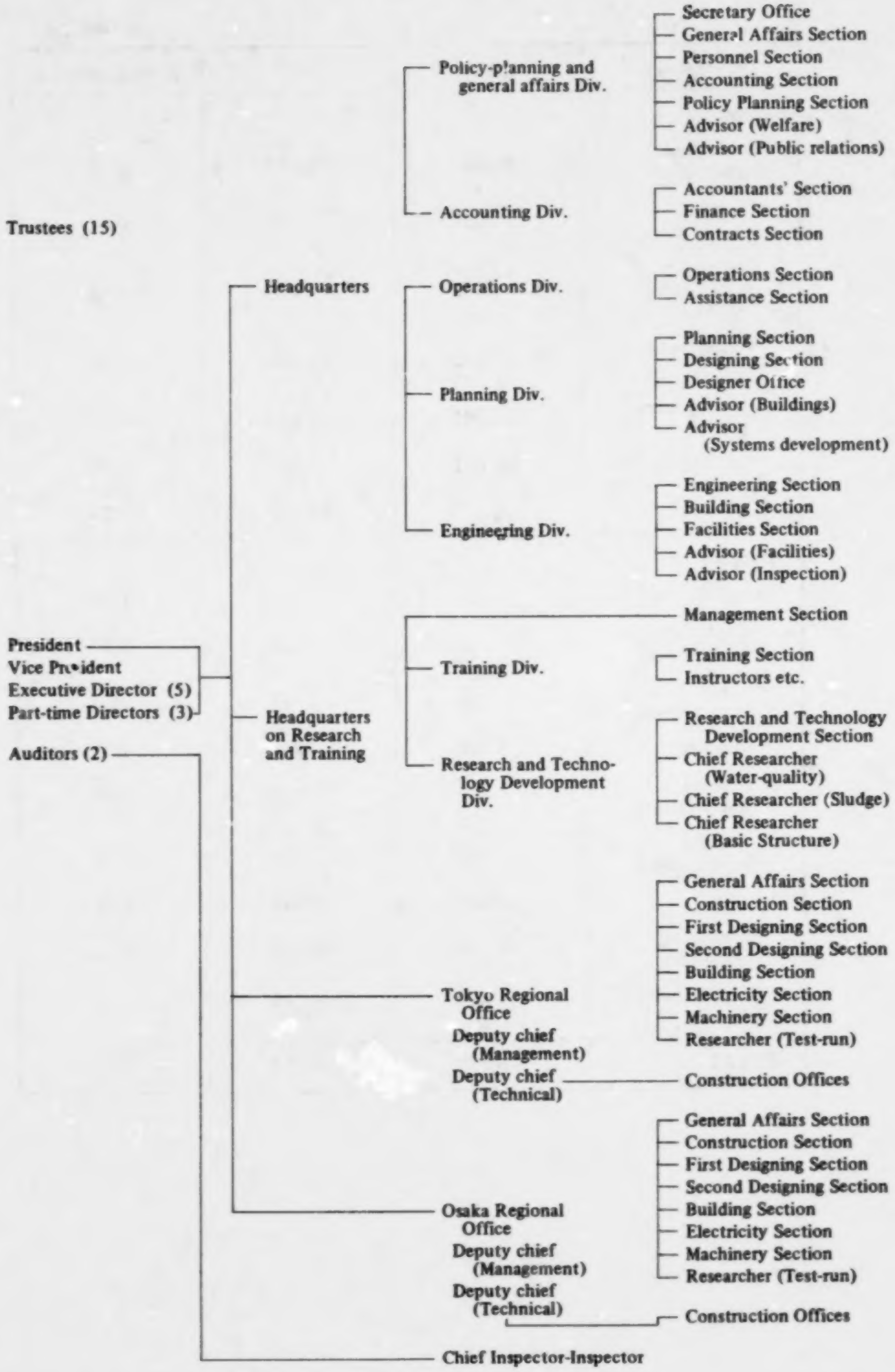

Table-12 Breakdown of Type of Work of Personnel in %

Clerical	Engineers						Total
	Civil/ Sanitary	Archi-tectural	Mechanical	Electrical	Chemists/ Biologists	Sub-total	
21.8	46.6	6.9	12.5	10.3	1.9	78.2	100

2. ACTIVITIES OF THE JAPAN SEWAGE WORKS AGENCY

2.1 CONSTRUCTION OF THE MAIN FACILITIES OF SEWAGE WORKS

2.1.1 Range of Consignment and Policy

Sewage works are composed of sewage treatment plants, pumping stations, trunk sewer, branch sewer and other collection system. Of these facilities, the construction of sewage treatment plants and pumping stations require highly advanced techniques involving civil & sanitary engineering, architectural engineering, electrical engineering, mechanical engineering and biological and chemical experitise supported by long experience. On consignment entrusted by the local governments, JSWA conducts construction work of the following main sewage facilities that do require such fully advanced techniques:

(1) Sewage treatment plant.

(2) Trunk sewer directly connected to the treatment plants.

(3) Publicly owned pretreatment plants other than the Sewage treatment plants described in (1).

(4) Pumping stations.

When the JSWA is entrusted by a local government, it gives top priority to the project of the water basin whether it concurs with the provisions of the environmental quality standards related to water pollution. Furthermore, immediately after completion of work and before commencement of operation, JSWA makes it a rule to conduct comprehensive starting-up operations to check and confirm the normal functioning and mechanical conditions of the total plant. The comprehensive start-up operations are followed by maintenance and operation work. The JSWA periodically dispatches engineers to the plant as an after care to offer proper technical guidance to the plant operators.

2.1.2 Cost

Concerning cost, the expenses are based on the standards stipulated by JSWA. It will be an aggregation of cost of each designed plan. Besides costs, administrative expenses will also be required. The administrative expenses are computed on a percentage basis as listed in Table-13.

Concerning payment, JSWA and the local government normally compile the payment terms after thorough consultation.

Table-13 Percentage of Administrative Expenses Accompanying Construction Work

Consignment Cost (million yen)	Percentage
below 500	5.3%
500 ~ 1,000	4.3%
above 1,000	3.3%

2.1.3 Methods of Work Execution

(1) Instructions governing basic plans

Prior to the construction work, JSWA generally conducts planning of schemes and specifications. But, even in this case, JSWA makes it a rule to submit the drafted plans for examination to the local government. And based on the instructions by the local government with regards the blueprints, the construction work is conducted. Therefore, the construction work by JSWA is based on the instructions by the local government.

(2) Work execution, supervision and inspection

JSWA invites tenders to execute work by contractors specializing in civil engineering, architecture, electrical engineering, mechanical engineering, etc. Then JSWA organizes a team of qualified engineers in civil engineering, architecture, electrical engineering and mechanical engineering to advance construction work by repeatedly conducting discussions. Stationed at the work site, the technical experts supervise the work based on JSWA's 'List of Contract Work Supervision.' These people are all experts with long experience in organizing and supervising sewage works construction from the Ministry of Construction, Prefectures and Municipalities, etc.

(3) Comprehensive start-up test operation

In coordination with the contractor, manufacturer and the local government, JSWA normally conducts a thorough start-up operation prior to actual maintenance and operation work of all units of the freshly completed sewage treatment plant including the sewage treatment, sludge treatment and pumping facilities. To guarantee continuous and proper functioning of all units, adjustments are made and trouble shooting performed. This comprehensive start-up run of facilities usually last for about 30 days concerning sewage treatment and for 10 days with regards pumping work.

(4) Financial procurement and temporary advances

Work to complete sewerage systems require a tremendous amount of funds. Although the main facilities of a sewerage system are given government subsidy, the amount of allotment is limited. However, the financing of preceding work must be tentatively prepared by the local government (subsidiary body) as advances. As JSWA considers it an important mission to cover construction of sewage works requiring prompt and urgent action, JSWA takes special financial measures to alleviate fund requirements of the local government concerned.

In other words, when JSWA is entrusted by a local government to construct a sewerage system, JSWA can acquire advances by loans from commercial banks the amount equivalent to the share to be borne by the local government concerned.

To alleviate the increasing financial requirements, measures can be taken to relieve the burdens of the local government to repay funds on a five-year deferred payment basis. This deferred payment method of five years has a grace period of one year repayable in eight different installments equalizing the incidence of interests for each six months.

2.1.4 Sewage Treatment Plants in Service Built by JSWA

Table-14 shows the construction projects managed by the Japan Sewage Works Agency.

Table-14 Construction Projects Managed by JSWA

	1972	1973	1974	1975	1976	1977	1978
Municipal Sewage Works	1	16	23	45	55	64	73
Basin-wide Sewage Works	0	0	3	7	8	11	16
Total	1	16	26	52	63	75	89

2.2 DETAIL DESIGN

2.2.1 Range of Consignment and Policy

In compliance with the requests by local governments, JSWA considers the construction of the main facilities of sewage works as her main business. Accordingly, JSWA is entrusted with the basic designing of the following main facilities. However, to accepting the consignment, the Agency makes it a rule to receive instructions in mapping out detail design.

(1) Sewage treatment plant.
(2) Trunk sewer directly connected to the treatment plants.
(3) Publicly-owned pretreatment plants other than the above sewage treatment plants.
(4) Pumping stations.

The detail design is deeply related to the actual construction work. Therefore, the detail design is generally mapped out as part of a single scheme that encompasses construction work. The engineering staff of JSWA may partly take part in the design drafting work which is mainly entrusted to private consultant engineering firms.

2.2.2 Costs

With regards costs to implement planning, they are computed on the standards stipulated by JSWA. The sewage treatment capacity, the pumping capacity and the length of trunk sewer are detrimental in determining the expenses directly needed. In addition, 10 percent of administrative expense is needed as contract fee.

2.2.3 Execution of Work

Upon signing the consignment contract, the work is implemented by a special team of experts of JSWA of civil engineering, architecture, mechanical engineering and electrical engineering organized in either Tokyo or Osaka Regional Office

Table-15 Sewage Treatment Plants in Service Built by JSWA

City or basin-wide	Construction starts	Service begins	Capital billion yen	Capacity 1,000 m^3/day
Motobu	Jan. 1974	Jul. 1975	3.5	6
Imabari	Oct. 1973	May 1976	4.1	34
Marugame	Oct. 1973	May 1976	3.7	17
Kasugai	Oct. 1973	May 1976	2.6	17
Kanuma	Oct. 1973	May 1976	3.1	17
Oyama	Oct. 1973	Jun. 1976	3.5	34
Sano	Oct. 1973	Jun. 1976	5.1	27
Nagaoka	Jul. 1973	Sep. 1976	4.3	28
Kure	Nov. 1975	Oct. 1976	1.0	23
Kitakyushu	Aug. 1975	Nov. 1976	1.0	50
Hohu	Sep. 1975	Jan. 1977	4.2	14
Higashimatsuyama	Oct. 1973	Apr. 1977	4.1	10
Yokohama	Jul. 1975	Apr. 1977	1.7	74
Ashikaga	Oct. 1973	Jun. 1977	4.5	24
Itako	Nov. 1973	Aug. 1977	1.4	2
Gamagori	Oct. 1973	Aug. 1977	5.7	56
Oita	Dec. 1975	Aug. 1977	2.3	12
Kawasaki	Aug. 1975	Sep. 1977	2.4	64
Machida	Jul. 1973	Oct. 1977	5.9	23
Kitakyushu	Sep. 1976	Oct. 1977	0.7	23
Fukuoka	Oct. 1975	Oct. 1977	1.6	10
Kyoto	Jan. 1976	Oct. 1977	2.2	40
Utsunomiya	Jun. 1974	Jun. 1978	8.0	36
Muroran	Jun. 1974	Sep. 1978	4.4	24
Saga	Aug. 1974	Sep. 1978	5.5	24
Kasumigaura	Aug. 1975	Oct. 1978	7.2	27
Kannonji	Nov. 1973	Oct. 1978	2.7	16
Mure	Oct. 1973	Oct. 1978	2.4	10
Numazu	Jul. 1974	Nov. 1978	3.4	16
Kawanoe	Jul. 1975	Jan. 1979	2.5	3
Nago	Jul. 1974	Mar. 1979	2.7	5
Ena	May 1976	Feb. 1979	1.6	7

depending upon the locality of the local government. The planning of trunk sewer is entrusted to the specialists in civil engineering of JSWA. However, prior to the planning work, the special project team confirms the wishes and views of the local government, carefully studies the contents of the approved project plan and makes on-the-spot investigations on the conditions of the work site before delegating the designing work to a private firm. In the meantime, in close coordination with the local government based on geological surveys, the scale of the plant, the model and type of the main equipment and the layout are decided as the basic plan which is forwarded to the Technological Committee in JSWA to confirm that no problems are involved and to assure the basic plan as flawless. This work is entrusted to the engineers in the Design Section of the Agency Headquarters. Then either at JSWA Tokyo or Osaka Regional Office, designing of specifications with regards the structure and machinery of each facility and the execution methods are conducted. And the completed and finalized plans are submitted to the local government. In this case, when deciding on the method of sewage treatment, the Design Criteria/Manual of JSWA serves as a guideline and, at times, pilot plant tests, prior to designing, become necessary. In this case, engineers in the Research and Technology Development Division of JSWA cooperate in the work to help incorporate the test results in mapping out plans.

2.2.4 Detail Design Mapped Out by JSWA

Table 16 shows the number of detail design mapped out by JSWA ever since its inauguration in 1972.

Table-16 Detail Design Mapped out by JSWA

	1972	1973	1974	1975	1976	1977	1978
Municipal Sewage Works	19	12	16	21	18	33	45
Basin-wide Sewage Works	0	2	4	7	4	8	10
Total	19	14	20	28	22	41	55

2.2.5 Evaluation of Detail Design

With regards detail design of sewage treatment plants, to make accurate evaluations on the observance of the contract by private consultant firms, evaluations are made in three stages of No. 1 (Time of completion of detail design), No. 2 (While construction is in progress), and No. 3 (Upon completition of start-up operation). And the evaluation of detail design is done. The results of the evaluation are recorded and filed to be reflected in the future and to perfect future designing work.

The evaluations of the sewage treatment plant design should be made in three stages: Evaluation by those who designed the plans. Evaluation by those engaged in construction stage concerning solicitude for convenience of execution. And evaluation from the standpoint of maintenance and operation.

Although depending upon the size, the construction work of a sewage treatment plant executed by JSWA normally takes between 5 and 7 years from the planning stage, construction work to comprehensive start-up operations. Therefore, the three stages of evaluation by a project team take more than 5 to 7 years.

Table 17 shows the outline of the three stage evaluation.

Table-17 Three Stage Evaluation of Design Work

	First Stage	Seond Stage	Third Stage
Time of evaluation	Upon completion of designing	During construction work. 3 years after commencing work on sewage treatment plant and two years after commencing work on sludge treatment plant.	Upon completion of comprehensive start-up test operation
Evaluation items	Evaluation by those in charge of designing plans ○ The results of drawings ○ Progress in work under commission	Evaluation by those engaged in construction work ○ Accuracy ○ Understanding ○ Workability ○ Adaptability ○ Material fitness ○ Estimate accuracy	Evaluation by those conducting trial tests ○ Consistency of each unit ○ Operation efficiency ○ Safety free degree ○ Maintenance
Evaluation made by	Plan designers Civil Engineering Architecture Mechanical Engineering Electrical Engineering Inspectors	Work supervisors Civil Engineering Architecture Mechanical Engineering Electrical Engineering Chief supervisor	Work supervisors Civil Engineering Architecture Mechanical Engineering Electrical Engineering Chief test operator

2.3 PROJECT PLANNING

2.3.1 Scope of Consignment and Policy

On consignment, JSWA commences project planning for sewage works (basic plan as stipulated in the provisions of Project Plan). In project planning for sewage works, the population in the drainage area and specifications of sewage treatment plants, pumping stations and their maintenance programs must be envisaged. Then they are incorporated into the Basic Plan of the planned sewage works project as an undertaking plan. JSWA then takes legal steps to have the sewage works scheme approved by the City Planning Law as it is related to city planning project and undertaking, and also by the Sewerage Law. JSWA, concerning necessary planning, compiles her undertaking plans in the following three stages. And depending upon the work scale of the scheme, these three stages of planning are completed within a year or more than two years.

(1) Basic survey

With regards the projected area of the sewage works, JSWA conducts surveys on the sewage flows and the conditions of the expected sites of trunk sewer, pumping stations and sewage treatment plants.

(2) Compilation of undertaking plan (for approval by the City Planning Law)

The total area of the expected draining area of the sewage works, location of the trunk sewer, a rough layout plan of the treatment plants, pumping stations and total sewer system length and other relevant items of sewage works that can be considered as part of city planning are prepared.

2.3.2 Procedures of Consignment and Expenses

With regards procedures of consignment, a local government first submits a paper requesting JSWA to construct sewage facilities on consignment. Then JSWA and the subject local government hold repeated discussions and conclude an agreement. As the expense to compile the initial planning of the project is not eligible to the government subsidy, the government or the requesting public entity must prepare for appropriate budget for the cost. The expense required to draft the project plan is computed on the bases decided by JSWA. The expense is inconsistent depending upon the scale of initial plan covering a wide range of subjects including Basic Survey, Basic Planning, Project Drafting, and Project Planning. However, the agency also demands expenses directly needed to compile outline of plan after surveying the geologocial and topographical conditions of the planned city area, the total area required for sewage treatment and other city conditions. Presently, most of the expense goes to private consulting firms, as they are entrusted to conduct the survey work and compilation of the outline of plan.

In addition, 10% of the total cost agreed upon is needed as administrative expenses.

2.3.3 Method of Implementation

When JSWA is entrusted by a local government to design plans, the work is undertaken by JSWA's staff in either Tokyo or in Osaka Regional Office depending upon the locality of the local government. And prior to the commencement of planning, JSWA's staff are dispatched to the local government to gather information on requests and desires and also to survey the actual conditions of the work site. A part of this work is often delegated to the consultants.

2.3.4 Project Planning Conducted by JSWA

Table-18 Project Planning Conducted by JSWA

	1972	1973	1974	1975	1976	1977	1978
Municipal Sewage Works	1	11	25	12	13	12	6
Basin-wide Sewage Works	1	4	1	1	0	0	0
Total	2	15	26	13	13	12	6

2.4 TECHNICAL ASSISTANCE

2.4.1 Scope of Consignment and Policy

JSWA offers technical assistance covering all fields of sewage works including planning, designing, scheming, constructing, maintenance and operations. The contents of the technical assistance offered by JSWA are as follows:

(1) Technical assistance regarding planning of sewage works

Besides analyzing and examining the contents of plans, JSWA offers technical advice and guidance. JSWA also engages in studies of advanced wastewater treatment, trouble-shoots problems of inundation, conducts surveys on water polluting conditions of rivers and researches on drafting a comprehensive basin planning for each drainage area.

(2) Technical assistance concerning sewage works design

JSWA not only offers technical advice and guidance upon examining plans for sewage works (structure, appropriateness of construction method and process, and estimates of costs listed in the plans) but also serves as a consultant for other aspects of sewage works.

(3) Technical assistance concerning sewerage work

Besides offering general advice and guidance with regards execution of work, JSWA conducts studies on construction methods when requested.

(4) Technical assistance concerning maintenance and operation of sewage facilities

JSWA offers general advice and guidance with regards maintenance and operation of sewage treatment plants, etc; technical diagnosis of operating and manipulating methods and water quality control; and consultation with regards maintenance and supervision systems.

(5) Other technical assistance

JSWA engages in consultative service by offering advices despite minuteness or roughness, fineness or coarseness of technical problems concerning operating system and other matters of sewage works. Furthermore, to positively engage in all problems concerning sewerage, JSWA dispatches personnel to offer advice and guidance to cities where prefectural government offices are located.

2.4.2 Consignment Procedure and Expenses

Concerning the procedure of consignment, it is the same as in the case of entrusting the designing of plans. However, in case of technical assistance, as the expenses are used to mostly cover administrative costs, they will be appropriated in the administrative account. The items in the account include salary for JSWA personnel while being dispatched to conduct on-the-spot studies of work-sites, their travelling expenses and other expenditures. Although depending on the contents of technical assistance, if the work involves planning of schemes, the expenses will be computed on the same basis as scheme planning. However, with regards simple consultation, it is normally free of charge.

2.4.3 Implementation Method

(1) Technical assistance by dispatching personnel

In compliance with the request rendered by a local government, JSWA dispatches her personnel to various work-sites to offer technical assistance.

(2) Technical assistance without dispatching personnel

With regards problems requiring technical research, the experts basically conduct survey at the Headquarters of JSWA or at her Regional Offices. But when necessary, JSWA personnels visit the work-site to conduct on-the-spot survey to collect materials and to consult with people concerned. Depending upon the content of technical assistance, at times, there are needs to involve consultants or specialists.

2.4.4 Inspection and Technical Service (After Care) at Sewage Treatment Plants

JSWA periodically dispatches her experts to sewage treatment plants constructed by the Agency and already in operation to inspect the operating and functioning conditions of the plants, to offer technical guidance as after care to the various local governments and to obtain materials that would be of help in the future designing works.

(1) Inspection and technical guidance
(i) Technical guidance concerning operation and maintenance
(ii) Servicing and trouble shooting of machinery
(iii) Matters to be fed-back to future designing of plant in general

(2) Inspection of maintenance & operation conditions
(i) Inspection of actual operating conditions of sewage treatment plant
(ii) Inspection of management and organization of sewage treatment plants

In 1978, JSWA will conduct periodic inspection of sewage treatment plants completed by the Agency in 7 cities that commenced operation in 1976 and in 7 cities that began service in 1977. However, together with the increase in the number of sewage treatment plants to be inspected in the future is expected. Therefore JSWA must cope with the situation by expanding the after care operation.

2.5 TRAINING

2.5.1 Outline

JSWA trains personnel belonging to the various local governments engaged in sewage work or those who are expected to engage in this field in the future. Furthermore, JSWA is driven by necessity to conduct highly efficient trainings to maintain and secure necessary number of engineers in the fields of civil engineering, architecture, electrical engineering and mechanical engineering to cater to all demands. According to Article 22 of the Sewerage Law, those engaged in drawing blueprints and those in charge of maintenance and supervision of sewage treatment plants require special qualifications as stipulated in the Law. The qualifications are given to only those who have completed the special training courses designated by the Ministry of Construction and the Ministry of Health and Welfare. Of all the trainings offered by JSWA, a number of these are those designated by the two Ministries. JSWA Research and Training Headquarters was built in Toda City, Saitama Prefecture with funds from both the government and various local governments. The scale of the Headquarters is as listed in Table-19.

Table-19 Outline of Main Building Housing
The Research and Training Headquarters

Address: 5141, Shimo Sasame, Toda-shi, Saitama-ken
Total land area: About 4,600 square meters
Structure of building

and number of floors: Reinforced concrete structure, 6 floors
Detail of each floor:
1st floor: Library, faculty room, reference room
2nd floor: Main office, water quality test laboratory
3rd floor: Classrooms for trainees, drafting room, auditorium, rest room
4th floor: Dining room, bath room, culture room
5th floor: Dormitory (38 rooms for 152 people)
6th floor: Rest room, wash room

2.5.2 Recruiting of Trainee

JSWA requests the mayors of the 10 large cities and the governors of all prefectural governments in Japan to recommend probable candidates for trainees. And from those recommended by the mayors and governors, JSWA carefully examines the past record and qualification of each candidate and selects trainees.

2.5.3 Training Expense

Each trainee bears the expenses for text books and food and accomodation during the training period. Other expenses are free of charge borne by JSWA.

2.5.4 Training Program

See Table 20.

Table-20 Subjects of Studies for Training

Course	Major	Trainees	Content	Result
Planning	Master Planning and Project Planning	Those chiefly engaged in work of preparing documents to acquire approval of project planning or improving sewage system in a city.	1. Basic matters & points in planning sewage facilities. 2. Studies on related laws and administrative acts. 3. Showing actual examples of planning work. (99 hours, 20 days)	Becomes person responsible for preparing papers to apply for approval and to offering guidance to consultants.
	Comprehensive Basin Planning	An official at either government or prefectural governments or municipalities engaged in comprehensive basin planning.	1. Basic planning of basin project. 2. Actual examples of planning. 3. Methods of water pollution analysis. (66 hours, 12 days)	Becomes person responsible for surveying basin project, and able to compile water plans for public use.
Designing	Detail Design – 1 –	Those who completed academic courses other than those related to civil engineering and who will newly be assigned to sewerage works.	1. Introduction to civil engineering, and basics of sewage engineering. 2. Designing and laying of branch sewer. 3. Surveying, flow rate measuring and design calculating. (162 hours, 34 days)	With the newly acquired civil engineering knowledge, becomes a person capable of designing branch sewer.

Course	Major	Trainees	Content	Result
Designing	Detail Design – 2 –	Those who completed civil engineering courses and who will newly be assigned to sewerage work.	1. Basics of hydraulics and soil mechanics. 2. Designing and laying of branch sewer. 3. Surveying, flow rate measuring and design calculating. (123 hours, 25 days)	With the newly acquired knowledge of basic civil engineering, becomes a person capable of designing branch sewer.
	Detail Design – 3 –	Engineers in other fields being transferred to sewage works.	1. Designing of sewage facilities. 2. Sewage engineering and planning of sewerage. 3. Problems in execution of plans. (99 hours, 20 days)	A person who can understand the contents of project planning.
	Trunk Sewer – 1 –	Having experience of over five years or equivalent.	1. Points on planning sewage system layout. 3. Points on soil mechanics. (99 hours, 20 days)	Be able to make decision of trunk sewer method. Able to instruct consultant while mapping out plans for execution.
	Trunk Sewer – 2 –	Engineers of supervisory class engaged in designing of trunk sewer.	1. Trends in Sewage Engineering techniques. 2. Recent Execution method and execution management. 3. Points on special methods and calculation. (99 hours, 18 days)	Be able to design, execute and instruct special methods.
	Treatment Plant – 1 –	Having experience of over 5 years or equivalent.	1. Selection of treatment methods. 2. Selection of sludge treatment methods. 3. Layout planning and capacity calculation. (99 hours, 20 days)	Able to make selection of engineering method and decision of basic designs at treatment plants. Also able to give guidance to consultants.
	Treatment Plant – 2 –	Those completing electrical and mechanical engineering courses or having experience of over 5 years or equivalent experience.	1. Designing of electrical equipments of sewage treatment plants. 2. Designing of mechanical equipments of treatment plants. 3. Specifications and computation of equip-	Able to make designing of electrical and mechanical equipment at pumping stations and treatment plant.

Course	Major	Trainees	Content	Result
			ment. (99 hours, 20 days)	
Designing	Treatment Plant – 3 –	Chief engineers in charge of designing or planning treatment plants.	1. Technical trends of sludge treatment at sewage treatment plant. 2. Points of designing sewage treatment plants & computation. 3. Countermeasures against industrial waste water. (57 hours, 10 days)	Able to plan layouts evaluate plans and give guidance of sewage treatment plants.
Management of Execution	Management of Construction Work	Those with over 5 years of experience or equivalent.	1. Planning and management of execution work. 2. Points on soil mechanics and work. 3. Disaster prevention and compensentation. Measures against local citizens. (99 hours, 20 days)	Be able to conduct on-the-spot supervisory work concerning work management.
Maintenance and Operation	Management of Maintenance and Operation – 1 –	Those newly assigned to sewage treatment plant.	1. Basics of sewage treatment plant. 2. Basic knowledge of water quality and actual examples. 3. Mock tests and machinery handling. (123 hours, 25 days)	Understand total system of operating sewage treatment plant to engage in maintenance work.
	Management of Maintenance and Operation – 2 –	Those having engineering experience in other fields and newly engaging in sewage treatment plant.	1. Maintenance of electrical and mechanical units of sewage treatment plant. 2. Symptoms and measures. 3. Safety management, water quality contol and countermeasures. (First half: 45 hours 10 days Practical teaching: 3 months Second half: 78 hours 15 days)	Understand total system of operating sewage treatment plant to engage in maintenance work.
	Management of Mainte-	Those over 5 years of actual experience or	1. Total management of sewage treatment plant and related laws.	Able to make maintenance and management of

Course	Major	Trainees	Content	Result
Maintenance and Operation	nance and Operation – 3 –	equivalent.	2. Water quality control and measures against accident. 3. Mock tests and machinery handling. (99 hours, 20 days)	sewage treatment plant.
	Total Management	Those expected to be head of a section or sewage treatment plant operator with experience on management, and supervision.	1. Total management of sewage treatment plant and related laws. 2. Water quality control and measures against accident. 3. Sewage management and management measures. (First half: 45 hours 10 days Practical teaching: 3 months Second half: 78 hours 15 days)	Person in charge of sewage treatment plant with leadership.
	Water Quality – 1 –	Those engaged in water quality test or those expected to be assigned.	1. Basic knowledge of sewage and sludge treatment. 2. Practical training of water examination test. 3. Water control practice. (123 hours, 25 days)	Able to analyze sewage and sludge at sewage treatment plant and also water quality control.
	Water Quality – 2 –	Those completing studies on biochemistry and to be assigned as leader of laboratory and industrial waste water monitoring.	1. Industrial waste water control and related laws. 2. Heavy metal analysis, etc. 3. Water control practice, etc. (99 hours, 20 days)	Able to understand mechanical analysis necessary for sewage treatment plant maintenance and to offer guidance concerning industrial waste water.
	Water Quality – 3 –	Section heads supervising industrial waste water monitoring.	1. Sewerage Law and Industrial waste water control. 2. Pretreatment plant and practical training. 3. Waste water control and administrative guidance. (45 hours, 8 days)	Becomes person in charge to control industrial waste water and inspect pretreatment plant.
Instruction and Guidance	Guidance – 1 –	(Prefectural employee)	1. Design criteria of sewage and planning.	Able to inspect and instruct work

Course	Major	Trainees	Content	Result
Instruction and Guidance		Prefectural governments' workers supervising work in related cities.	2. Standard application for approval, and problems related to application for subsidiary. 3. Points concerning construction method and supplementary work. (99 hours, 20 days)	to apply for approval and subsidiary.
	Guidance – 2 –	(Managerial class) Those in managerial class in sewerage or equivalent.	1. Sewerage problems and financing and management. 2. Pollution measures of public waters and trend. 3. Problems involving execution of sewage works. (33 hours, 6 days)	Understand the points with regards sewerage operation to offer guidance to personnel.
	Guidance – 3 –	Those in the managerial class being transferred to sewerage work.	1. Problems related to sewerage and administrative financing and management. 2. Explanation of sewerage facilities. 3. Problems involved in execution of sewage work. (66 hours, 12 days)	Thoroughly understand sewerage undertaking.

(Notes) 1. The number of years is based on high school graduates.
2. Those having experience means sewerage experience.
3. Engineering experiance means technical engineering experience in all fields.

2.6 ENGINEER CERTIFICATION

2.6.1 Certification System

JSWA performs certification examinations to those aspiring to become qualified engineers to engage in designing of sewage treatment facilities and to those to become authorized managers and supervisers engaged in the construction work of such facilities. This certification system was inaugurated with the aim of encouraging capable engineers in other fields such as water supply, river, harbor, etc. to transfer or being introduced into the sewerage field by enabling them to obtain certificates stipulated in Article 22 of the Sewerage Law in a short period of time.

2.6.2 Contents of examination

The method of engineer certification is based on examinations in academic subjects. The contents and level of examinations for 1st class and 2nd class engineers are decided by the Ministry of Construction. And the contents and level of examinations for 3rd class engineers are decided by both the Ministry of Construction and the Ministry of Health and Welfare. On October 9, 1975, notification with regards definite contents of examination was issued as shown in Table-21.

To ensure fairness in the examinations, the preparation of examination papers and judgement of results are conducted by five members of the Engineer Certification Committee nominated by the President of Japan Sewage Works Agency.

Table-21 Academic Subjects and Standards for Engineer Certification Examinations

Classification	Subject	Examination Standard	Examination Method
First Class Engineers	Sewerage Planning	To have necessary knowledge to map out plans concerning layout, structure and capacity of sewer.	Descriptive and selective
	Sewage Treatment Plant Planning	1. To have general knowledge of mechanical and electrical equipment, their capacities and their structures installed in the sewerage system or at sewage treatment plants. 2. To have necessary knowledge to calculate strength of structure of plant. 3. To have necessary knowledge concerning construction methods of sewerage facilities. 4. To have general knowledge concerning specifications of designs of sewage facilities.	Selective
	Management Methods of Work Execution	To have general knowledge concerning mapping out of plans for execution work of sewerage, supervising of work process, material quality control, safety control, etc.	
	Sewage Disposal	To have general knowledge with regards sewage and sludge.	
	Related Laws	To have general knowledge concerning related sewerage laws.	
Second Class Engineers	Sewerage Planning	1. To have general knowledge of mechanical and electrical equipment, their capacities, and their structures installed in the sewerage system or at sewage treatment plants. 2. To have necessary knowledge to calculate strength of plant. 3. To have necessary knowledge concerning construction methods of sewerage facilities. 4. To have general knowledge concerning specifications of designs of sewage facilities.	Selective
	Management of Work Execution	To have general knowledge concerning mapping out of plans for execution work of sewerage, supervising of work process, material quality control, safety control, etc.	
	Sewage Treatment	To have a rough outline of knowledge with regards sewage and sludge treatment and disposal.	

Classification	Subject	Examination Standard	Examination Method
Second Class Engineers	Related Laws	To have a general knowledge related to sewerage laws.	
Third Class Engineers	Sewage Treatment	To have necessary knowledge in sewage and sludge treatment and disposal.	Selective
	Industrial Waste Water	1. To have general knowledge with regards the influence of industrial waste water to public sewer. 2. To have general knowledge of function of pretreatment plant.	
	Maintenance and Operation	To have necessary knowledge concerning maintenance and operation of sewage treatment plants and pumping stations.	
	Safety Control	To have general knowledge concerning safety control of sewage treatment plants and pumping stations.	
	Related Laws	To have general knowledge concerning laws and acts related to sewerage.	

2.7 RESEARCH AND TECHNOLOGY DEVELOPMENT

2.7.1 Aim

It is important to reinforce research and technology development activities of JSWA to fulfill its mission. Especially, it is strongly urged to introduce new technology that are more economical, stable and safe for practical use. To cope with these demands, JSWA has founded its Research and Technology Development Division to conduct studies on the following four fields:

(1) Surveys to develop new technology.
(2) Surveys to perform technical consultations with regards designing, planning and building of sewage facilities.
(3) Surveys to evaluate new technology.
(4) Special surveys in compliance with requests by government and local governments.

2.7.2 Facilities

The main facilities and experimental facilities and devices employed by the Research and Technology Development Division are as shown in Table 22.

2.7.3 Evaluation of New Engineering/Equipment

Japan is an attractive market for sewage equipment manufacturers. Many new products are recently introduced and many companies claim avantages of their products or process. Their advertisement like a kaleidoscope. This creates an additional confusion to not only local personnels but also governmental administrators. Therefore, proper evaluation of the newly introduced engineering or equipment is required.

Many campanies want that their products are evaluated. Selecting ones among

Table-22 Tests and Research Facilities and Devices

	Major Facilities	Major Testing Apparatus and Analyzer
Research and Training Center	(Main Laboratory) Laboratory for Chemical Analysis Laboratory for Instrumental Analysis Balance Room Constant-temperature Room Bioassey Laboratory Computer Room Data Room	High rate chromatograph Atomic absorption analyzer T.O.C. analyzer Calorie meter Automatic BOD analyzer CN analyzer Automatic nitrogen analyzer Photometer Gas chromatography
	(Annex building) In-door Experimental Station Sludge Pilot Station Greenhouse Mobil Laboratory (Otsu City)	TOC/TOD automatic monitoring instrument Secondary treatment pilot plant X-ray fluorescence analyzer Eutrophication test tank Automatic control unit for pilot plant Wet oxidation pilot plant Pylolysis pilot plant
Others	Lake Biwa Advanced Waste Pilot Plant Laboratory (Semimentation, sand filteration and activated carbon facilities) Composting Unit (Sakado city)	

many of them is a hard job. Therefore, the subject to be evaluated are selected from the requests of national government or municipalities. The priority in the selection is determined by JSWA. Research works for the evaluation (to see how the equipment performs) are managed by Research and Development Division.

For the purpose of making fair evaluations or uneven conclusions, a special committee is engaged in this work. It is named "Committee for Evaluation of New Technology". Members of the Committee are as follows:

	number
Professors of universities:	3
Engineering chief director of sewage works in municipalities:	4
Engineering administrator of national government:	1
Research director of Public Works Research Institute:	1
Executive director of JSWA:	1

Conclusions given by the Committee may be refered when the national government determines whether the equipment/engineering can be subject to the construction grant, and if yes under what circumstance.

At present, the following topics are under way of the evaluation:

(1) Pure oxygen activated sludge process.
(2) Best selection of anti-air pollution equipment for sludge incinerator.
(3) Economical and technical feasibility of automatic control for activated

sludge process.

(4) Rotating biological contactor process

2.7.4 Current Research Topics and Findings

2.7.4.1 Evaluation of New Thickening and Dewatering Equipments for Sewage Sludge

(1) Purpose

The prime importance in sludge treatment is how to decrease sludge volume. Any costs associated with sludge handling and disposal process are entirely dependent on moisture concentration. A gravity thickener does not work satisfactorily in many cases. Therefore recently power drive thickening equipment appears to attract attentions. They are floatation or centrifuge.

Such new developments as screw press or belt press are introduced and claim a high performance in dewatering, in this year, their performance were compared side by side in the field.

(2) Results

By using the advanced dewatering devices, 10 different sludges were applied to see their performance.

(i) A belt-press type dewatering machine has the characteristics that the high belt speed dewaters more amount of sludge, but moisture concentration in final cake is high.

(ii) A pressure-filter dewatering machine has the characteristics that wider slit opening and shorter pressing time increase the capacity at sacrificing solid concentration.

(iii) The Belt press achieved 96% SS recovery.

(iv) With Cationic polymer at dosage rate 0.6 ~ 0.8%, dewatering performance of the belt press was good.

2.7.4.2 Evaluation of Sewage Sludge Incinerator

(1) Purpose

At present, multiple hearth type, fluidized bed type and rotary kiln type are populare for the purpose of municipal sludge incineration. The target of this research is that performance of the three alternatives are compared in several ways in actual field.

Thus recommendation is provided to the potential users.

(2) Finding

(i) The actual sludge fed rates and moisture content in the sludge never meet design value. They always exceeded the design or expected values.

(ii) Usually more diluted sludge is fed, and therefore much auxial fuel is required to help burning.

The following is an example of least fuel consumption among the surveyed.

Type	Capacity	Moisture	Fuel	Fuel consumption
i) multiple hearth	300 t/D	78.5%	A oil	130 kg/t-D.S.
ii) rotary kiln	24 t/D	61.2%	coal gas	180 Nm^3/t-D.S.

(iii) Odor components are removed through a process of thermal decomposition. The fluidized bed type incinerator need not after-burning system has an advantage in the mean.

	Type	Capacity	Moisture	Fuel	Fuel consumption
i)	multiple hearth	50 t/D	79.3%	A oil	410 kg/t-D.S.
ii)	fluidized bed	40 t/D	83%	A oil+kerosene	320 kg+820 kg/t-D.S.

(iv) The required air rate concerned with fuel consumption, depended on type of incinerators. The least air rate is more desirable. The following were examples.

	Type	Total air rate	Fuel
i)	multiple hearth	2.2 ~ 3.5	A oil
ii)	rotary kiln	2.3 ~ 3.2	coal gas
iii)	fluidized bed	1.4 ~ 1.8	A oil

(v) Any incinerator described above did not have difficulties in operation and monitoring. The rotary kiln type incinerator has the merit that we can see dry section and burning section of incinerator, if city gas is supplied as auxial fuel.

(vi) Continuous incinerator operating is best for economy, but in the field, this practice was rare. For intermittent operation, fluidized bed type is most suitable due to short warming up time and high heat holding capacity.

(vii) Because calory which is sent out by exhaust gas is major portion of the total energy losses, we must reuse it as much as possible.

2.7.4.3 Development of Index for Predicting Eutrophication Potential

(1) Purpose

AGP (Algal Growth Potential) in effluent has been known useful to indicate eutrophicational potential in natural water course. Our purpose of this study is to develop a standard procedure for AGP examination.

(2) Results

"Algal Assay Procedure" for fresh water, saline water and sea water has been developed. The procedure discribed in the report is consisted of (1) Principle, (2) Apparatus, (3) Test Algae Species, (4) Synthetic Algal Nutrient Medium, (5) Method of Sampling, (6) Method of Stock Culture, (7) Method of Cultivation (Temperature, Illumination, Rate of Oscillation and Amount of Inoculum etc.), (8) Parameters of Algal Growth (Maximum Specific Growth Rate, Maximum Standing Crop).

2.7.4.4 Design Criteria for Covered Sewage Plant

It is not seldom that a sewage treatment plant is a next door to a school, houses, or downtown due to land shortage. This situation enforces municipalities to design fully covered sewage treatment plant.

To the neighbors, the cover may provides good appearance or a nice play ground. However, to the plant operator it may give miserable working environment, if exhaust system does not work properly.

Problems being associated with the covered plant will be pointed out through this study. The followings are findings in this year.

(1) Disadvantage

a. Not only construction cost but also operational cost of the covered plant are higher.
b. Expantion or improvement of covered one is much difficult.
c. Installing large machineries and taking out them are not easy.
d. High humidity, insufficient light, or bad smell are very common, resulting in poor working environment.

(2) Improvement for working environment

A double covers is a popular practice to improve the nuisance environment caused by the primary deck. If the primary setting tank and secondary aeration tank are covered with a inner deck; and the fumes are vented, the problems are desolved.

(3) Exhaust goes from the covered space; especially thickeners and sludge dewatering building, smell bad. Several practice for deodering are applied. Among them, soil adsorption is in high potential for this purpose.

2.7.4.5 Best Selection of Unit Process for Treating Wastewater for Small Communities

The unit processes which are operated in large municipalities are not always successfully operated for small communities. These communities do not afford skillful plant operators. Their plants are usually suffered from the operational problems which are associated with sharp and wide variation in influent rate.

A special consideration should be paid to selection in unit process for them.

A foolproof but still high performance is desirable. In this year, specific biological process were surveyed to see how they meet the requirement for the small plant. The following two are found potential alternatives to the traditional sludge process.

(1) Pure Oxygen activated sludge process

The process was less subject to sludge bulking than the activated sludge process.

A large variation of influent flow rate causes the sludge bulking in the conventional activated sludge process. The Oxygen system is as designed to fully automatically operate that the smaller number in operators and less experience personnels can manage it.

(2) Rotating biological contactor

Rotating the contacting media is all thing to do for wastewater treatment. Operation itself is quite simple. On the other hand, effluent quality is not so good as that from air activated sludge. Fine flocs can not be removed in a final clarifier. Due to the flocs, the final effluent does not appear sparkling clear neither high transparency. In addition to this the effluent BOD does just clear 20 mg/ℓ at best. The improvement of this is under the study. Nevertheless, the process is high potential to the small communities.

2.7.4.6 Evaluation of Equipment for Sewage Plant Automation and Development of New Technology

– Automation for Sewage Treatment Plant –

Facilities for constant MLSS and MLDO control are equipped in the pilot plant in JSWA's laboratory.

2.7.4.7 Application of Advanced Construction Methods to Specific Cases of Sewer, Pumping Station and Sewage Plant Placing

The purpose of this study is to standardize technical and economical feasibility of such advanced methods for sewage system construction as stabilization of foundation and foundation work.

For this purpose many construction sites are visited and all aspects of the construction are summerized. In this survey, our major concerns are:

(1) Geographical and foundational condition on construction site.
(2) Stabilization of foundation.
(3) Landslope protection wall and excavation and their auxiliary construction methods.
(4) Foundation works.
(5) Environmental disruption produced by construction and its countermeasure.
(6) The problem awaiting solution on construction method of sewerage systems.

This study just begings and results are coming up now.

2.7.4.8 Management of Industrial Pre-treatment

(1) Purpose

This research program has two purpose, one of which, is to show the most suitable operational management requested for the fulfilment of the industrial pre-treatment facilities. The other purpose is to standardize the reference by which municiplaties can judge whether proposed pre-treatment facilities are acceptable or not.

(2) Finding in 1977

(i) Monitoring industrial wastewater

Generally speaking, companies do not like to pay much efforts to their wastewater handling since it is negative investment.

They neglect the watch or monitor effluent quality, which is discharged to publicly owned sewer. However, on the other hand, if the wastewater is reused for some purposes in the plants, they try their best to operate the treatment facilities. Municipalities or local authority should pay attention in introduce a policy in which the companies are induced to have recycle system.

(ii) Operation of pre-treatment facility

In general, a selection of unit pre-treatment process is found reasonable for almost all the cases. Activated sludge system is a good choice for such food industrial wastewaters as soybean paste product or brewery industry. Pressurized floatation process is a better application for paper recovery industrial wastewater than chemical precipitation. The pre-treatment plant which were surveyed performed satisfactorily.

(iii) Characteristics of industrial effluent

Characteristics of typical industrial wastewater are summerized from the data in the field survey.

Tile industry; contain such heavy metals as Pb, and much SS
Sanitary toilet bowl industry; high alkaline concentration
Landry and dry cleaning industry; high pH, BOD, oil

(iv) Sludge disposal

Sludge produced from the organic wastewater pre-treatment process is rich organic content and converted easily to fertilizer after composted.

2.7.4.9 Operating and Improving Existing Sewage Plant (III)

(1) Inflow equalization

Almost common problem appears in air mixed flow equalization tank is mixing efficiency goes down when the water level lowers, which results in the deposition of solids. Sometimes several diffusers located at entrace of the basin were buried under the solids.

(2) Sludge bulking prevention

Dosing alum to the aeration tank was quite effective to cure bulked sludge into normal one. After getting back to normal, there is no necessity to dose continuously alum but one or twice a week.

(3) Improved over-loaded aeration tank by marox system

Conventional activated sludge plants have been suffered from serve sludge bulking due to specific carbon hydrates included in Orange canning industrial wastewater.

During the canning season the final clarifier did not separate bioflocs. SVI was over 1000. Marox pilot plant had been operated by side. It showed very good performance. Aeration time was 1/3 of the air system and much better BOD removal resulted. Marox's economy did not differ much from air system.

2.7.4.10 Improvement of Sludge Handling Process

It is a common practice among many sewage plants that side streams generated within sludge handling process are recycled to the secondary facilities. There are many cases that the poor quality of the side stream retarded the secondary performance. Our survey showed that all most all the thickeners did not perform so satisfactorily as designers intended, which resulted in poor supernatant quality. In the thickeners, sludge became septic and floated up; that is most common phenomenum seen in many thickeners. The following interrelationship between suspended solid concentration in the sludge and time until floating up are found.

$$\frac{1}{t} = a \times Ts + b$$

t: time until floating up (hr)
Ts: overall solid concentration (%)

The equation says that diluted sludge is less subject to floating. Our experiment showed that dilution with secondary effluent improved sludge settability but not retarded the underflow concentration. Polymer had effects on preventing the sludge floating but not on improving underflow concentration chlorine, at 50 ~ 100 mg/liter, prevents the sludge from floating.

2.7.4.11 Recovery Energy and Raw Material from Sewage Sludge

(1) Purpose

Under a condition of a little oxygen, a high temperature, and a certain pressure, sewage sludge is decomposed in a different way from a traditional incineration. This is a so-called "pylosis or thermal decomposit" process. End products from this process are high caloric fuel gas, reuseful raw material and inert ash. It will be a main process for disposing sewage sludge, if the reaction goes efficiently. We commenced to power much research effort to see what are key factors in good performance of the pyloysis.

The specific research purpose on this subject was to find optimum process parameters for wet thermal decomposition and dry type pyloysis.

(2) Dry type pyloysis

(i) Caracteristics of fed sludge cake
moisture content 83.1%, volatile matter 82.3%
calorie potential 4715 kcal/kg, carbon content 43.3%

(ii) Yielding rate of tar and residual ash

Temp (°C)	Tar	Residual Ash
600	14 g/kg-sludge cake	102 g/kg-sludge cake
700	30	60
800	26	53

(iii) Specific surface area of residual ash was 1.1 m²/g at 600°C, 1.2 m²/g at 700°C, and 2.5 m²/g at 800°C

(iv) Caloric value of tar was 4500 ~ 8000 kcal/kg and carbon content of tar was about 50%

(v) Caloric value of gas was 2500 kcal/m³ at 700°C and 3500 kcal/m³ at 800°C

(3) Wet thermal decomposition

The higher temperature and more air existed in the reactor, more volatile solid is removed through the process.

When the process of the wet thermal decomposition is operated at 250°C and hold for one hour, carbohydrate contained in the fed sludge is completely oxidized to carbondioxide and water. The protein is decomposed to organic acid which are relatively stable, and fat is degradated to fatty acid.

2.7.4.12 Application of Sewage Sludge to Agricultural Use

(1) Purpose

The purposes of the research are

(i) to obtain the optimum conditions for composting sewage sludge

(ii) to investigate the moving phenomena of sludge-borne heavy metals in soils and crops

(2) Findings

(i) The following optimum conditions were obtained for composting centrifuged sludge cake

1) Mixing ratio (Wet weight base)
Rice hull: Sludge cake = 0.3 ~ 0.4 : 1
Rice-straw: Sludge cake = 0.15 ~ 0.3 : 1
Both were composted without adding the returned compost.

2) Moisture
Rice hull mixture: less than 65%
Rice-straw mixture: less than 75%

3) Detention time
Fermentation: 1 ~ 2 weeks
Curing: 2 ~ 3 months

4) Aeration rate
Rice hull mixture: $250\ \ell/m^3$ min.
Rice-straw mixture: $150\ \ell/m^3$ min.

5) Turning
Fermentation: every 1 or 2 days
Curing: every 1 or 2 weeks

6) pH
As pH of centrifuged sludge cake ranged 5 ~ 7, it assumed that pH had no bad effect on fermentation.

(ii) The following indexes are assumed to be useful for composting centrifuged sludge cake.

1) Fermentation: BOD, Odour
2) Curing: Carbon, Ignition Loss, Calories, Δlog K, Fumic Materials

(iii) Sludge borne isotope labeled heavy metals were used to investigate the moving phenomena in soil and crops.

1) Only a little of isotope labeled heavy metals were eluviated into soil from sludge cake added; e.g. 1.2% of sludge-borne ^{73}As and less than 0.1% of sludge-borne ^{109}Cd, ^{203}Hg, ^{51}Cr and ^{65}Zn were eluviated.
2) Amount of sludge-borne heavy metals incorporated into berley was also a little; e.g. less than 0.1%.

2.7.4.13 Survey of Emission from Sewage Sludge Incineration Process

(1) Purpose

At present there are 50 cities which operate incinerators to burn sewage sludge. Many municipalities are going to have ones. Public concern to air pollution is very keen. It is afraid that stack exhaust gas from the incinerators is a new air pollution source. The main target of this research is to see how it gives impact to atomospheric environment and, if yes, how to decrease it.

(2) Findings in 1976

From last 2 years surveys, generally, quality of the exhaust gas from the sewage sludge incinerators were fond to meet local emission standard. If electoric precipitators were applied, the particle concentration can be reduced as low as

1/10 of the standard. However, such advanced exhaust gas treatment operation as EP results in a increase in operation cost. Therefore a new engineering should be developed to concentrate to:

(i) decrease in exhaust gas volume
(ii) decrease in NOx

(3) Findings 1977

In order to decrease the exhaust gas rate, the least air supply is desirable. Without any defficiency, minimum air requirement is tried to find out. All over air ratio for multiple hearth incinerator is 1.3 and for fluidized bed is 1.2. Under the least air supply, also NOx concentration is not high.

At the same time, several methods were tried to remove NOx. Among them, "selective homogeneous gas-phase method with NH_3" was found very effective for this purpose. More than 75% NOx could be removed.

2.7.4.14 Experimental Study for Establishing Design Parameters of Lake BIWA Tertiary Treatment Plant

(1) Alum precipitation

(i) Recycling one portion of precipitated alum sludge to the flush mixing tank results in great improvement of solid settlability in the clarifier. The improvement is about 80%. When apply this practice, the overflow rate of the clarifier can be enlarged at 1.8 time for the same SS removal efficiency.

(ii) If the precipitated sludge is treated at pH 3 with sulfic acid before recycled, virgin alum requirement can be cut down to a half.

(iii) Pressure floatation can thicken the clarified underflow sludge, up to 3 ~ 4% solid concentration, while a gravity thickener does only 1% at best.

(iv) 80 ~ 90 percent alum can be recovered through selective ionexchange process. The alum is as effective to P-precipitation as virgin one.

(v) A belt press dewatering process can reduce moisture concentration of the sludge down to 80%if properly conditioned with polymer.

(2) Sand filteration

(i) The maximum allowable flow rate for the dual mixed media filteration is between 200 ~ 300 m^3/m^2/day. A greater rate than this causes phosphorous leakage to effluent.

(ii) Suspended solid capture through the filteration bed is 2.5 ~ 2.8 kg/m^2.

(iii) Back washing of the dual media filter is not perfect without air buffle agitation.

(3) Activated carbon adsorption

(i) COD adsorption capacity is around 40 ~ 50 g/kg AC when COD removal rate is less than 50%.

(ii) Flow rate, SV10 is for up flow and SV4 for down flow.

2.7.4.15 Performance of Unox System in Kyoto City

Since the Fall 1977, a unox plant has operated in Kyoto City for treating mixture of dye industrial wastewaters and domestic sewage. The plant has a capacity of 40000 cu-meters a day. It is the biggest unox plant in publicly owned ones in Japan. The whole sewage plant are completely housed in fancy and good looking building, since it is built in mid residential area. Thus nuisancefree operation is possible; free from noise, smell and vibration.

At BOD aeration tank loading 2 kg/m³/day, BOD removal rate is 1.8 kg/m³/day. BOD in final effluent is 18 mg/ℓ at the loading.

2.7.4.16 Performance of Jet Aerator (Impingment Type) in Deep Aeration Tank

The city of Kitakyushu had to increase suddenly a capacity of Hiagari Sewage Plant 2 times as much as that of the original design after the air blower room and main air supply pipes had been completed. There was no available space left over to extend facilities to cope with the increased quantities, so the only alternatives were to deepen the aeration tanks and improve the oxygen transfer efficiency without increasing air supply equipment. The jet aerator was tried to answer the needs.

The following is average data for its performance:

oxygen utilization rate	20%
kWH/kg.BOD removed	1.08

As compared with traditional diffuser system, air supply requirement to keep a sufficient DO concentration is about a half. Power requirement to remove unit BOD is about equal in the both system. The nozzle holes seem subject to clogging and errosion for a long use. At present no data on this matter are available.

2.7.4.17 Oder Removal of Vented Air from Sewage Treatment Plant
– Soil Filteration –

Exhaust gas from several sources in a sewage plant is introduced to several difficult soil filter beds for the purpose of fume removal preliminary findings are as follows:

(1) A soil bed which contains high concentration of organic matter and has large void spaces shows good performance.

(2) Moisture content in the bed is critical for optimum operation.

(3) Original odor content 5000 ~ 17000 unit are reduced to 50 ~ 130 unit after applied to the soil filter. The removal rate is 98 ~ 99%.

2.7.4.18 Solidification of Sludge Cake with Cement

Public concern to environmental pollution is too keen that even sanitary landfill practiced in many where is not practical in and around Tokyo Metropolitan area. Fishermen are against sludge ocean dumping even though it is stabilized by an anaerobic digestion. The sludge cake should be treated with some means before final disposal regardless it is the sanitary landfill or ocean dumping.

One approach to this is to solidify the sludge cake by blended with ash or cement. Tokyo Metropolitan Government goes on this line for reclamating seashore by sewage sludge application. Before going deep to this way, Tokyo asked JSWA to examine how the solidified sludge cake were stable to outer stress and could keep such toxicants as heavy metals off leakage. Finally how this approach are economically feasible in our main interest.

Findings

(1) Portland cement along could not improve characteristics of the sludge cake to meet the Tokyo's request.

(2) When ash and alminum cement are blended to the sludge cake, the final products are satisfactory.

Sludge cake (%)	Ash (%)	Al-Cement (%)	One day strength (kg/cm^2)
83	12	5	0.56 – 0.83
80	10	10	1.09 – 2.92
70	10	20	1.10 – 6.76

2.7.4.19 Infiltration-Inflow Survey in Sewer Systems

Purpose

Infiltration-inflow often causes hydraulic overload for sewer systems, which results in dry weather overflow to streams or wet weather inflow increases in separate systems. The purpose of this survey is to accomplish the countermeasure for infiltration-inflow problem. As the first step, surveys were carried.

Findings

(1) Peak infiltration rate appeared in August or September and recorded from 45.5 to 409 m^3/liner 1000 m/day (or from 7.9 to 92.5 m^3/ha/day)

(2) A total amount of rain water included in sanitary sewer was 1 ~ 6% of total storm water.

(3) Poor joint between two pipes, manhole wall and inverts poorly constructed caused infiltration. In addition to those, other sourses of inflow are manhole covers and poor house drain systems.

(4) Countermeasures for infiltration-inflow are following:
  a. Check of house drains and correct of cross connection
  b. Use of pipes with water tite joint for example PVC pipe, clay pipe with compressive joint and concrete pipe with high precise socket joint.
  c. Chemical grout on leaky sewer section.
  d. Upgrade of construction methods, that is, laying and bedding.

2.7.4.20 Process Design of RBC for Koya City Treatment Plant

Koya City is a famous religions city.

A secondary facility in the new treatment is planed to have RBC. A pilotplant operation concludes the following parameters; at 4°C.

Retention Time	2.8 Hr
Hydraulic Loading	50 litters/sq-meter/day
BOD Loading	5.2 gram/sq-meter/day

2.7.4.21 Process Design of RBC for Shirakaba Treatment Plant

The Shirakaba is a typical resort lake where a new sewage plant is to be built. The local ordinance requests that the new treatment plant should discharge final effluent which meet the following qualities: BOD is 18 mg/ℓ and SS is 10 mg/ℓ. The requirement of 10 mg/ℓ for the suspended solid in the effluent is hard to

keep. Therefore specific attention is paid to selection of unit process. Primary clarifier. RBC and rapid sand filtration are the result.
Planning data are as follows:

	Water quality (mg/ℓ)			Removal efficiency (%)		
	Primary effluent	RBC effluent	Filtrate	RBC	Filtration	RBC + filtration
BOD	126	26	18	78	31	86
SS	104	15	10	86	33	90

A RBC pilot operation resulted in the following design parameter:

Retention Time: 1.4 Hr
Hydraulic Loading: 120 litter/sq-meter/day
BOD Loading: 15 gram/sq-meter/day
at temperature 4°C

2.8 TECHNOLOGICAL COMMITTEE

At JSWA, to deliberate, investigate and survey important matters relevant to technology of sewage works, a special technological committee is organized at the headquarters. The committee is composed of executive directors in charge of planning, engineering works and research and technology development; heads of the planning, engineering works, research and technology development, and training division; chiefs and deputy chiefs of both Tokyo and Osaka Regional Office. The chairman of the committee is the Vice President of JSWA.

The committee is entrusted with the responsibility to perform the following investigations and surveys:

(1) Concerning detail design criteria.
(2) Concerning inspection of construction work and its criteria.
(3) Concerning design of projects requiring judgement of high technical engineering level.
(4) Concerning application of new technology.

To expedite the committee's deliberations of the above mentioned matters expertly, the following sub-committee are held:

(1) Design Criteria
(2) Execution Criteria
(3) Project Case Study

Presently three university professors are participating in the committee discussions to survey exclusive technical matters.

Of the various undertakings by JSWA, concerning new techniques to be put to practical use for the first time or new techniques that are already put to practical application but pending evaluations of their practicability as sewage treatment plant equipment involving sewage treating methods, treating facilities and instruments, the committee plans to positively introduce and evaluate such new techniques and foster practical application in an orderly manner.

When the survey results prove the necessity of practical application of new engineering techniques, the committee will designate a demonstration plant to conduct experiments. In this case, the committee will decide the basic policy of the application plan and other important matters for the demonstration plant.

The all round appraisement of the operating results the experiments at the demonstration plant will be put together by the committee.

Presently, the design criteria/manual by the Design Criteria Sub-Committee divided into civil engineering, architecture, mechanical engineering and electrical engineering is bringing about good results.

And the committee activities with regards introducing new techniques are going well.

3. THE FUTURE OF JAPAN SEWAGE WORKS AGENCY

As aforementioned in "The Background for the Establishment of Japan Sewage Works Agency," JSWA was founded as a group of highly specialized engineers to supplement the insufficient engineering of each local government to cope with the rapidly increasing demands to improve its sewerage system. And it was also a countermeasure to meet the environmental quality standards in rivers, lakes and coastal waters all over Japan with regards water pollution.

Since establishment, JSWA has been widely participating in all phases of activities including designing, constructing, research, technology development and engineer training in the field of sewerage works. Catering to the increasing demands expected in the future, JSWA will be obliged to continue to expand its activities.

Presently, the 4th Five Year Sewerage Construction Plan (7,500,000 million yen) is being implemented. As the Water Pollution Control Law was partly revised, the Discharge Elimination System in closed water areas such as the Seto Inland Sea, the Ise Bay and the Tokyo Bay is enforced after one year of preparation period. In the near future, discharge elimination of phosphorous is to be regulated newly.

The Seto Inland Sea, the Ise Bay and the Tokyo Bay border important land areas containing large cities and strategic industrial zones. Therefore, sewerage measures to be implemented in these areas are drawing keen attention as an important domestic political problem. These areas are regarded as the core of industrial and business activities deeply related to our daily lives of Japan where water consumption is tremendously high. And since water shortage is anticipated in these areas, it is most urgent to not only prevent their water from pollution but also to develop methods that would most effectively reuse and recycle effluent from sewage treatment plants as important water supply. Therefore, sewerage measures in these areas must be dealt with in close cooperation with the nation, prefectural governments and municipalities in harmony.

Experts predict that the nation's sewerage undertakings will confront with the problem of reorganization. In that event, it will become an important task with regards the position and role JSWA should take.

In closing, JSWA is destined to take on more responsibilities to carry on sewage works in Japan in the coming years.

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WASTEWATER AND SLUDGE RESEARCH PROGRAM

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ABSTRACT

In the early 1970's, EPA was heavily involved in advanced wastewater treatment (AWT). It was believed that physical-chemical treatment would be needed to solve water quality problems that could not be eliminated by biological secondary treatment. At that time there was an environmental movement sweeping our country to cleanse our surface water of all pollution. The focus was predominantly on point sources. The present prevailing attitude is to use conventional facilities for removal of conventional pollutants (BOD and suspended solids) and reliance on pretreatment for removal of toxic and unconventional pollutants. Thus, our research funds are being spent on sludge disposal on land, toxic substances removal, and improving existing treatment plants to make them more effective and less costly to operate and maintain. In recent years our sludge management research has been switched from energy intensive processes, such as incineration, to obtaining useful products, such as compost, liquid and gaseous fuels. Our present projects are associated with the problems concerned with land disposal of sludge and the conversion of sludge into useful products. In the past, considerable sums of money were spent on nutrient removal and lagoon upgrading. Today, we have several processes that can be used for removal of phosphorus and nitrogen. They appear to be adequate for the time being. The outcome of our lagoon program has been that today there is more confidence both in the capabilities of conventional pond systems and in the use of supplementary devices to upgrade the existing lagoons. The research of the Wastewater Research Division is continuously changing to be responsive to regulatory changes, needs of the wastewater treatment community and to shifting world situations.

WASTEWATER TREATMENT PROCESS DEVELOPMENT

Major efforts in the past have been devoted to the development of oxygen aeration, lagoon performance, control of the nutrients nitrogen and phosphorus, uprating of existing treatment processes and disinfection. Current efforts are on approaches that show potential savings in energy and resources. Novel and innovative treatment concepts will receive more emphasis in the future. Some of the U.S. Environmental Protection Agency's (EPA) current interest in wastewater treatment process development are discussed below.

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DEEP SHAFT PROCESS

An EPA demonstration grant to evaluate an innovative biological treatment concept has been made to the City of Ithaca, New York. The concept is the Deep Shaft Process, the heart of which is a vertical, activated sludge reactor consisting of two concentric pipes, an inner downcomer leg and an outer riser leg. The process was initially developed and piloted in Billingham, England, by Imperial Chemical Industries (ICI).

Compressed air is injected into the Deep Shaft downcomer at about one-third total depth to provide the necessary oxygen for aerobic metabolism. The different liquid densities thereby created in the two legs cause mixed liquor to continuously circulate up the riser and back down the downcomer. Due to the high pressures existing at the bottom of the shaft, the saturation concentrations of the gases in air plus respiration by-product, carbon dioxide, are increased many fold at this point. Consequently, supersaturated dissolved gases, primarily nitrogen and carbon dioxide plus a small amount of unused oxygen, come out of solution as they travel up the riser leg to the surface. The natural flotation effect which results from this phenomenon has led to the development of two alternatives for liquid/solids separation. They are the standard gravity clarification with a degassing standpipe interposed between the settler and the reactor, and flotation clarification with polymer addition. Both modes of clarification will be evaluated at Ithaca. The inventors of Deep Shaft claim savings in power, volume, overall treatment cost and less excess sludge production than conventional activated sludge. Most certainly the land requirements will be less, making it applicable to industrial plants and municipalities with limited space.

The plant at Ithaca is designed for 760 m^3/day (200,000 gal) with a detention time of 38 minutes in the flotation clarification mode. The shaft will be 0.5 m x 133 m (18 in. x 435 ft.). Parameters to be evaluated in addition to process performance, will include power consumption, excess sludge production, and the economic tradeoff between the larger reactor volume required in the gravity clarifier mode versus polymer requirements in the flotation mode. Power consumption is tied directly to oxygen uptake rate and oxygen utilization. Due to the combination of high intensity mixing and greatly elevated pressures, oxygen transfer rates up to 2,885 kg/m^3/hr (180 lb/1000 ft^3/hr) and oxygen utilizations of 90 percent or more are reportedly achieved, yielding an effective power transfer rate in wastewater of 2.4-3.6 kg O_2/kWh (4-6 lb O_2/hp-hr). By comparison, conventional air aeration systems typically achieve 5-15 percent oxygen utilization and 0.6-1.2 kg O_2/kWh (1-2 lb O_2/hp-hr) power transfer rate. The potential economic savings in energy are considerable if the above figures for Deep Shaft power consumption can be verified.

The unusual cycling environment created by rapid circulation in the shaft is credited with stimulating high microorganism respiration rates at the expense of cell synthesis. This phenomenon has led to claims of potential reductions in excess biological sludge production compared to conventional activated sludge regimes. Based on pilot plant experiences to date, it is anticipated full-scale excess sludge production will approximate 0.5 kg TSS/kg BOD removed at F/M loadings of 0.75-1.0 kg BOD/day/kg MLVSS. Historically, sludge production at loading rates of this magnitude would be expected to approach 1.0 kg TSS/kg BOD removed. Because of the huge capital and operating costs involved in sludge processing and disposal, this aspect of Deep Shaft operation also suggests significant potential economic impact.

STANDARD FOR EVALUATING OXYGEN TRANSFER

As the USA continues to search for more effective ways to conserve energy, greater emphasis is being placed upon the development of energy efficient processes in pollution control technology. The evaluation of the efficiency of oxygen transfer devices and systems will assume greater importance as engineers seek less energy intensive equipment. In spite of the considerable effort devoted to oxygen transfer technology, it is evident that unanimity of opinion has not been achieved in the development of a standard procedure for the evaluation of oxygen transfer devices. The areas of disagreement lie not only in the details of conducting oxygen transfer tests, but also in the methods of data evaluation.

Presently, manufacturers rely on clean water shop tests (i.e., performance tests conducted in a test tank at the manufacturing plant), for describing the oxygen transfer capability of aeration equipment. The chemical constituents of wastewater vary substantially from one source to another. The concentration of surface active agents present greatly affects the Alpha factor. Temperature, tank geometry, and bubble size are other factors which influence the prediction of oxygen transfer in wastewater or mixed liquor based on clean water test results. A further complicating feature of this is that oxygen demand in activated sludge systems is due to a respiring biomass, whereas in the clean water test it is chemically induced.

Faced with this problem, many consulting engineers have written requirements for field performance testing into their specifications for aeration equipment. Contractors and manufacturers recognize this liability in preparing bids and will include larger factors of safety; thus, many aeration systems will be adequate but overdesigned. Some systems, however, may still prove to be inadequate.

All of the above discussion emphasizes the dilemma that the design engineer, the municipal decision maker, and the regulatory grant reviewer face in comparing alternative oxygen transfer systems. With increasing energy costs stimulating extensive interest in fine-bubble diffusion devices for the first time in this country, it has become imperative to develop uniform methods for testing and evaluating oxygen transfer which accurately assess equipment capabilities and which through repeated usage would assume the role of a consensus standard. A unified approach to evaluating competitive aeration systems would benefit all involved; the designer and municipal official in making trade-off decisions and selecting the equipment best suited for a specific situation.

A 2-year research grant has been awarded to the American Society of Civil Engineers (ASCE) to develop a tentative standard for evaluating oxygen transfer capacity of aeration and oxygenation equipment in water and wastewater. The ASCE has established a Subcommittee on Oxygen Transfer under the Committee on Environmental Standards. This Subcommittee is chaired by Dr. William Boyle, University of Wisconsin. The final report for the project will contain a tentative standard, literature review, and recommendation for verifying the tentative standard in the field.

As an adjunct to developing a standard for evaluating oxygen transfer, an EPA contract has been awarded to Los Angeles County Sanitation District to compare seven submerged air aeration devices. The transfer devices will be tested under strictly controlled conditions in a 6 m x 6 m x 7.5 m (20 ft. x 20 ft. x 25 ft.) tank filled with clean water. The diffusers to be tested will include:

1. Fine bubble diffuser system designed for total floor coverage. (Norton)
2. Fine bubble diffuser system designed for side wall installation. (FMC, Pearlcomb)
3. Static mixer aerator system. (Kenics)
4. Jet aerator system. (Penberthy)
5. Coarse bubble diffuser system variable orifice. (Bauer)
6. Coarse bubble diffuser system fixed orifice. (Sanitaire)
7. Coarse bubble diffuser system designed for total floor coverage (Envirex)

Data generated on the test program to date suggest that fine bubble systems can substantially reduce energy requirements. The second phase of this project will evaluate the more promising systems under field conditions in activated sludge systems. The field studies will determine the extent that operation in wastewater (i.e. mixed liquor) decreases the oxygen transfer efficiencies observed with fine bubble equipment in clean water. The study will also determine whether the fine porous media of some of these devices are subject to rapid headloss buildup and unacceptable maintenance requirements for cleaning.

ACTIVATED BIO-FILTER

The activated bio-filter (ABF) system is a unique combination of conventional biological processes. Settled activated sludge instead of being returned to the head of the aerator as in other two-stage systems where an attached growth unit precedes an activated sludge unit, is first recycled to the top of the tower for mixing with primary effluent and passage through the redwood media biocell in trickling filter fashion. There is no intermediate clarifier. Biocell effluent containing the recycled settled sludge and sloughed humus is then fed directly to the aeration tank.

Carrying active suspended biomass through the biocell reportedly greatly increases the organic loading and removal capacity of this unit. The recommended design organic loading for the biocell of 3.2 kg/day/m^3 (200 lb BOD/day/1000 ft^3) is 4-7 times higher than for high-rate stone media trickling filters and 2-4 times higher than for synthetic media trickling filters. BOD removal for all three of the above at their respective design loading rates is approximately 65 percent. Due to the high-removal capability of the biocell, advertised nominal detention time requirements for the downstream aerator to complete secondary treatment are minimal, ranging from 1/2-1 hour depending on the actual residual BOD concentration from the biocell.

The ABF process as now marketed qualifies as an efficient high-rate biological treatment system. Broad acceptance and deployment of the technology is lagging due to the lack of full-scale confirmation of the claims made for it. Data at full-design loading exist only at pilot scale. Municipal treatment plants which have installed ABF systems are in the early part of their design lives and are typically operating at 1/3 to 1/2 of stipulated capacity.

Comprehensive cost estimates of biological treatment systems recently completed by Culp/Wesner/Culp indicate that at the manufacturer's recommended design conditions, the ABF process is substantially more cost effective than any of the established pure attached growth biological processes and is cost competitive with established suspended growth processes at all sizes up to 4.38 m^3/sec (100 mgd). Substantiation of the manufacturer's pilot results on a large-scale municipal installation would stimulate greater consideration of a potentially cost-effective innovative treatment alternative.

A grant has been awarded to the City of Helena, Montana, to collect and evaluate full-scale operating and performance data on ABF process under carefully controlled loading conditions up to the manufacturer's recommended design organic loading. Helena's current influent flow is 16,650 m^3/day (4.4 mgd). Although Helena's current biocell loading is only one-half of design, the unit's flexibility permits sections to be removed from service such that several loading rates can be imposed.

WASTEWATER DISINFECTION DEVELOPMENT

The U.S. Federal Water Pollution Control Act Amendments of 1972, Public Law 92-500, clearly established the responsibility of the Environmental Protection Agency (EPA) for reducing and controlling the pollution of navigable waters. The Act established as a National goal that the water quality should provide for the protection of fish, shellfish and wildlife and for recreation in and on the water by July 1, 1983. The principal mechanism used to achieve the objectives of Public Law 92-500 was by implementing effluent standards for industrial and municipal wastewater dischargers. Included in the effluent standards were biochemical oxygen demand, suspended solids, pH, and fecal coliform bacteria. The limit on fecal coliform bacteria was set at 200 per 100 milliliters. This requirement generally necessitated the use of separate, continuous disinfection processes in addition to the biological or physical-chemical processes for BOD and suspended solids removal.

On July 26, 1976, the EPA deleted the fecal coliform requirement from the effluent standard. Reliance on the site-specific water quality standards of each State to set disinfection requirements for municipal wastewater treatment plants is now the practice. Thus, disinfection will still be required by State regulation, but no uniform Federal requirement has to be achieved. Some of the reasons for the change in the regulation are: (1) Toxic effects on the aquatic environment of residual chlorine, (2) halogenated organic compounds are potentially carcinogenic, (3) continuous disinfection of wastewater is not needed in certain locations, (4) potential dangers associated with the use of elemental chlorine, and (5) the amendment of the effluent standards provides flexibility in disinfection requirements. Cost was not considered as a main factor, because public health and protection of aquatic environment are the overriding considerations affecting any disinfection requirements. The unnecessary and excessive use of disinfectants, however, is wasteful both in terms of available financial resources and energy.

The adverse environmental effects of residual chlorine in chlorinated wastewaters has prompted EPA to search for alternative disinfection agents which would satisfy the disinfection requirements and be environmentally acceptable. The present alternatives currently under investigation are: chlorination-dechlorination, ozonation, chlorine dioxide and application of ultraviolet light.

PILOT PLANT DEVELOPMENT OF OZONE DISINFECTION

Ozone is a potent oxidizing agent, and its reaction with oxidizable material is non-selective. The demand exerted by organic matter in effluents can have a marked influence on the disinfection efficiency and reliability of ozone. Care must be exercised in making certain that the ozone produced is utilized in the most efficient manner; otherwise, the operating costs of ozonation may be needlessly high due to excessive use of energy resources. One of the main items influencing the cost is the contactor.

Private industries concerned with supplying ozone generation facilities have as their objective the most cost-effective package. The package normally includes air preparation, ozone generation, and ozone contacting. Every conceivable liquid-gas contacting device is marketed with ozone systems. EPA has initiated at its pilot plant at the Robert A. Taft Laboratory, Cincinnati, Ohio, a project to determine the advantages and disadvantages as well as the economics associated with the contactor. This study has completed evaluating three generic types of contactors commercially available. It is intended to evaluate more in the near future. The three contactors evaluated were: (1) positive pressure injector (PPI), (2) a packed column (PC), and a bubble diffuser (BD).

All the contactors were designed and fabricated in EPA's pilot plant. The packed column contactor was a 230 mm diameter glass column packed with 3.1 m of 12 mm ceramic Intalox saddles. Secondary effluent entered the top of the column and exited at the bottom. The ozone entered at the bottom and flowed countercurrent to the secondary effluent. Residence time of the secondary effluent was 20 seconds at a flow rate of 75 l/min. The pressure injector was a PVC gas-liquid contactor into which both secondary effluent and ozone gas were pumped under a positive pressure of 50 to 60 kPa. The gas and liquid mixed in a specially designed head compartment, then flowed concurrently downward through an inner cylinder. Upon reaching the bottom of the inner cylinder, the gas-liquid mixture reversed flow and flowed upwards through a larger cylinder. At the top the gas and liquid separated, the gas flowing to the exhaust gas line and the liquid flowing downwards through a second concentric cylinder. Total liquid residence time in the system was 20 seconds at 75 l/min. The bubble diffuser contactor consisted of three aluminum columns, each 3.7 m high and 300 mm in diameter, connected in series by PVC piping. The three columns were staggered vertically so that secondary effluent which had been pumped to the top of the first column, could flow by gravity to each of the two successive columns. The ozone enriched gas was injected through domed ceramic diffusers located at the bottom of each column. The first column received 50 percent of the total gas flow, while the other two columns received equal fractions of the remaining 50 percent.

Testing of the different contactor types was performed using a split plot design where whole plots were arranged in a balanced incomplete block design. The whole plots were arranged in a balanced incomplete design due to the physical restriction that only two of the contactors could be set up in parallel at any given time. Thus, two contactors per day were tested with both contactors receiving the same applied dose at any given time. The order of the dose levels was balanced so that each dose level was used first, second, and third in a day the same number of times for each contactor. Randomization consisted of randomizing the order in which the pairs of contactors were compared.

The first set of experiments evaluated the percent ozone utilization as a function of applied ozone dose. The results, as shown in Table 1, indicated that overall percent ozone utilization was highest in the bubble diffuser, followed by the packed column, and then the positive pressure injector. As the dose was increased, percent ozone utilization in all contactors decreased. In Table 2, the total coliform log reduction (TCLR) in the above experiments are summarized. The data reveal that overall mean TCLR was higher in the bubble diffuser than in either the packed column or the positive pressure injector. The overall mean TCLR in the latter two contactors was not significantly different.

The second set of experiments was run to determine the change in percent ozone utilization and microorganism reduction when the applied dose was varied by changing the ozone concentration and maintaining a constant gas to liquid flow ratio. In Table 3, the mean percent ozone utilization data per contactor at each dose level are summarized. The results indicated that overall percent ozone utilization was higher in the bubble diffuser than in the packed column or the positive pressure injector while percent utilization in the latter two contactors was similar. The rate of decrease in percent ozone utilization at higher doses was substantially less in the bubble diffuser than in the other two contactors. The total coliform log reduction data are shown in Table 4. The overall TCLR in all contactors was similar. Although the overall utilization of ozone in the bubble column was higher, only 0.5 mg/l more ozone was absorbed than in the other two contactors. This increase was not sufficient to cause a marked increase in TCLR.

TABLE 1. MEAN PERCENT OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSAGE (INCREASING Q_G/Q_L, CONSTANT Y_1)*

Dose mg/l	Mean Percent Ozone Utilization (% U) (Standard Deviation) Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean % U, All Contactors
3.3	77 (2)	85 (4)	90 (2)	84 (6)
6.7	53 (3)	58 (2)	86 (2)	66 (15)
10.0	41 (3)	46 (3)	81 (3)	56 (18)
Mean %U, All Doses	57 (16)	63 (17)	86 (5)	

*Q_G = carrier gas flow rate
Q_L = liquid flow rate
Y_1 = concentration of ozone in the carrier gas

TABLE 2. MEAN TOTAL COLIFORM LOG REDUCTION (TCLR) AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Q_G/Q_L, CONSTANT Y_1)*

Dose mg/l	Mean Total Coliform Log Reduction (Standard Deviation) Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean TCLR All Contactors
3.3	2.90 (0.36)	2.85 (0.36)	3.12 (0.10)	2.96 (0.31)
6.6	2.94 (0.44)	3.13 (0.10)	3.56 (0.25)	3.21 (0.38)
10.0	3.04 (0.34)	3.31 (0.11)	4.52 (0.55)	3.62 (0.75)
Mean TCLR, All Doses	2.96 (0.36)	3.10 (0.29)	3.73 (0.69)	

*Q_G = carrier gas flow rate
Q_L = liquid flow rate
Y_1 = concentration of ozone in the carrier gas

TABLE 3. MEAN PERCENT OZONE UTILIZATION AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_G/Q_L)*

	Mean Percent Ozone Utilization (Standard Deviation)			
Dose mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean % U, All Contactors
1.5	84 (4)	79 (3)	89 (2)	84 (5)
4.5	68 (5)	68 (4)	80 (4)	72 (7)
7.5	62 (4)	62 (3)	77 (3)	67 (8)
Mean % U, All Doses	71 (10)	69 (8)	82 (6)	

*Q_G = carrier gas flow rate
Q_L = liquid flow rate
Y_1 = concentration of ozone in the carrier gas

TABLE 4. MEAN TOTAL COLIFORM LOG REDUCTION (TCLR) AS A FUNCTION OF APPLIED OZONE DOSE (INCREASING Y_1, CONSTANT Q_G/Q_L)*

	Mean Total Coliform Log Reduction (Standard Deviation)			
Dose mg/l	Positive Pressure Injector	Packed Column	Bubble Diffuser	Mean TCLR, All Contactors
1.5	0.81 (0.68	0.73 (0.59)	0.56 (0.40)	0.70 (0.54)
4.5	3.07 (0.43)	2.90 (0.39)	3.24 (0.49)	3.07 (0.44)
7.5	3.28 (0.40)	3.37 (0.30)	4.00 (0.49)	3.55 (0.50
Mean TCLR, All Doses	2.39 (1.25)	2.34 (1.25)	2.60 (1.58)	

*Q_G = carrier gas flow rate
Q_L = liquid flow rate
Y_1 = concentration of ozone in the carrier gas

DEMONSTRATION OF ULTRAVIOLET LIGHT DISINFECTION

Ultraviolet has been demonstrated to be feasible under certain conditions, but reliability is hampered by wastewater quality and inadequate design. Most UV units available on the market are designed for treating potable water. It is only recently that equipment manufacturers have focused their attention on wastewater.

EPA has funded a full-scale demonstration project on ultraviolet disinfection at Waldwick, New Jersey. A prototype ultraviolet system is being tested on a 30,000 m^3/d (8 mgd) conventional activated sludge effluent to obtain cost, reliability of achieving the desired coliform reduction, maintenance requirements and electrical power needed. The ultraviolet unit which will be used for this project is a prototype system, the UVS 5000, manufactured by the Uvoxx Company, Jersey City, New Jersey. The unit has been specifically designed for treatment of municipal wastewater effluent. It is especially flexible in operational modes and thus well suited to a research oriented study. It is characterized by a body having a series of spaced, parallel, quartz-jacketed mercury lamps (400 in number) extending from end to end. The placement of the lamps has been engineered to utilize a thin film concept with a minimal head loss through the unit. Actual water thickness between quartz jackets is 6.4 mm (0.25 inch). The water flows perpendicular to the UV lamps to an exit tank, where air which is used to ventilate the UV lamps is injected. This air contains a small amount of ozone. To insure unimpeded transmission of ultraviolet radiation through the wastewater, the unit is equipped with an automatic wiper mechanism adjustable for stroke frequency. The close tolerance wiping mechanism is designed to prevent scale buildup and solids accumulation on the quartz sleeves, but its performance has never been evaluated. The high intensity, short detention time concept needs also to be evaluated. A dimmer control will be incorporated into the UV unit so that electrical energy input can be varied without changing detention time. This could eventually act as a dosage control device. For example, as wastewater quality varies, changes in UV intensity would send signals to the dimmer to increase or decrease the current supplying the lamps. Finally, ozone generated in the ventilation air will be injected in the exit holding tank to provide additional disinfectant action. Since the purpose of the ventilation air is to provide the proper temperature for optimum lamp efficiency, the amount of ozone produced will vary with the volume of air being circulated, which will, in turn, vary with seasonal temperature changes. These effects will be closely monitored throughout the project.

Results obtained thus far indicate that approximately 600 UV lamps would be needed to achieve satisfactory disinfection for a design flow of 30,000 m^3/d (8.0 mgd) and a peak flow of 60,000 m^3/d (16 mgd). The total cost of ultraviolet light disinfection computes to approximately \$0.008/m^3 (\$0.03/1000 gal). This is a very preliminary estimate, but certainly indicates UV to be competitive with chlorination.

UV does not add chemicals to the wastewater, and the dose use for destroying bacteria is so small, changes in chemical compounds present will be minimal. Bioassay tests with rainbow trout support this hypothesis. Ultraviolet radiation may prove to be an alternative where wastewater quality is high and its volume is low.

CONTROL OF TOXIC COMPOUNDS

WASTEWATER TOXICITY

No discussion of wastewater would be complete without mentioning the concern for the presence of toxic metals and organics, because most wastewaters are a variable mixture of domestic and industrial wastewaters. The Environmental Protection Agency has established a list of 65 priority toxic pollutants under Consent Decree. The Agency is under court order and legislative mandate (Section 307 b and c of the Clean Water Act Amendments) to establish standards for these organics for 21 industries. The Office of Water Programs plans to spend more than $3 million on contracts to survey municipal wastewater and sludges for the priority pollutants. Unfortunately, the existing analytical protocol for organics has poor sensitivity in raw wastewater and sludges. The Wastewater Research Division has a grant with the University of Washington to improve the protocol and conduct a sampling survey of 25 municipal wastewater treatment plants. The grant with The University of Washington at Seattle, Washington, started June 1, 1978, and will end June 1, 1980. The amount of EPA funding is $400,000. The first task will be to develop and verify analytical methods to determine toxic priority pollutants in domestic wastewaters and sludge at a concentration of less than 1 µg/l. Attempts to apply the presently available industrial protocols to domestic wastewaters have failed because of poor detection limits. For many of the priority organic compounds, the present limits are 30-50 µg/l. This is far too large a limit to be of practical use in a survey of municipal treatment plants. After the test methods have been developed, surveys will be conducted of 25 municipal plants for toxics (129 priority pollutants) in the influents, after primary treatment and in the effluent discharge before and after chlorination, if practiced. Sludges will also be analyzed. A variety of plants containing various compositions of industrial/domestic flows and treating the wastewaters by a variety of processes will be surveyed. From this there will emerge the beginnings of a data base which will provide information on influent occurrence and concentration, removals currently being obtained by a variety of treatment processes and an evaluation of the toxics in sludges. Certain compounds tend to accumulate in sludge 10 to 1000 fold higher in concentrations than the influent. A corollary, yet extremely important objective of this study is to develop and verify the analytical methods that will be used in the survey. Surrogate tests for toxicity of the effluents, such as Ames test and acute fish toxicity will complete the study.

A data base on the occurrence and distribution of metals is being developed. The University of Tennessee has a grant to locate and tabulate existing data on influent heavy or toxic metals to POTW. To date, 47 percent of the cities contacted have stated that data exist that have not been reported. All these data will be evaluated to determine the reliability and quantity of the data. A concerted effort is being made to determine the source of these metals and what percent is due to domestic source, runoff, and industry. The grant at The University of Tennessee at Knoxville, Tennessee, started October 10, 1977, and will end August 23, 1979. EPA's contribution is $142,788. Metals discharged to a treatment plant have a special significance since, in the absence of proper precautions, they may be cycled from one medium of the environment to another; since metals cannot be destroyed, they can only be contained. Thus, removal from the liquid flow in a plant merely concentrates the metals in the sludges.

A survey of the City of Kokomo, Indiana, is being conducted by Purdue University on a $137,000 grant from EPA. The City of Kokomo has a 64,350 m^3/d (17 mgd) activated sludge wastewater treatment plant that receives a variety of industrial wastes in addition to the regular domestic waste. The grant started in November of 1977 and will be finished in September 1979 when the final report is published.

The study will determine sources of toxic metals and cyanide in the local municipal wastewater and sludges and establish a general protocol for identifying, quantifying and regulating such toxics in other municipal wastewaters. The objectives are to be accomplished through a comprehensive sampling and analysis program for stormwater, residential, industrial, and total municipal wastewaters and sludge. Elements of interest are those of greatest significance in land spreading of municipal sludges or the potential damage to the aquatic environment. They are Cu, Ni, Cd, Zn, Hg, Pb, Cr^{+3}, total Cr and cyanide.

A 60-day heavy metals mass balance around unit operations of the municipal treatment plant is nearing completion. High overall removals of metals, ranging from 68 percent for Ni to 97 percent for Cd, are being obtained through the plant which consists of activated sludge followed by tertiary filtration of effluent. These high removals are believed to be due, in part, to discharges of pickle liquor to the sewer from a steel plant next to the wastewater treatment plant. The iron which often produces a dark brown coloration of the wastewater is an effective precipitant for trace metals.

An evaluation of the Zimpro sludge reactor and thickener showed that relatively little total mass of metal was released from the sludge solids to be recycled to the plant in the supernatant.

There is a marked decrease in the total mass of metals entering the plant during weekend periods particularly for Cd, Cr, and Ni, pointing out the probable industrial origin of these pollutants. Sampling of feeder sewers containing only residential wastewaters had shown very low concentrations of Cd, generally less than 0.5 μg/l. Five of the six major trunklines serving Kokomo have been sampled to date.

Since the main thrust of EPA's efforts for toxics control is source control and pretreatment before discharge to a municipal treatment plant (POTW), it is not likely very much of our research funds will be devoted to process modification to enhance removal of toxics. Under ideal regulatory conditions POTW should not have any toxics in the influent; however, experience has shown that toxics do enter treatment facilities from diffuse sources. Presently, two approaches are concerned with toxics. They are the use of activated carbon in the aerator of an activated sludge system and the treatability of toxic substances.

One of the potentially useful processes for control of toxic organic compounds is adsorption on activated carbon. It has the merit that the compounds are destroyed when the carbon is thermally regenerated. Adsorption is a complex process which involves both the nature of the carbon surface as well as the characteristic of the molecule. Some of the latter factors include solubility, molecular weight, polarity, ionization, orientation at the surface, and more. All of these factors contribute to the adsorbability of a compound which, operationally, can be determined quantitatively by determining a batch equilibrium adsorption isotherm. During the last year the adsorption isotherms for 60 organic compounds were determined. These were collected into a simple publication entitled "Carbon Adsorption Isotherms for Toxic Organics," Environmental Research Laboratory, Cincinnati, Ohio, 45268, May 1978.

While carbon is highly effective for the removal of organics, it is important to point out that the variation of loading on carbon is great. For example, the adsorption of 60 compounds mentioned above varied from 0-360 mg/g of carbon at an initial concentration of 1 mg/l of compound. Relatively small changes in a molecule can alter the adsorbability of the compound. For example, unsubstituted benzene is barely adsorbable, 0.7 mg/g. Substituting an OH group for one of the H in the benzene molecule increases the adsorption by a factor of 30. When Cl is substituted, this factor increases to 133.

In summary, adsorption (or non-adsorption) depends on many factors, not the least of which is the substitution on a parent molecule. It is one of the goals of the research on treatability to discover those factors which govern adsorption and ultimately to be able to predict adsorption in some systematic way. Without this capability, laboratories will be burdened with the need to evaluate adsorption for thousands of compounds.

The treatability of selected toxic compounds is being studied in EPA's physical-chemical wastewater treatment pilot plant. Five compounds are being added to the raw sewage prior to treatment by alum clarification, duel media filtration and granular activated carbon. The first five compounds to be tested are: dimethylphthalate, ethylbenzene, carbon tetrachloride, trichloroethylene, and nitrobenzene.

Automatic composite samplers, designed to avoid loss of volatile organics, collect samples at appropriate locations in the system. The samples are analyzed by gas chromatographic procedures. Preliminary data indicate that, from an influent averaging 120-180 μg/l of each compound, reductions across the alum-polymer clarification unit range from 11 percent for dimethylphthalate to 66 percent for ethylbenzene. Some of the volatile substances are undoubtedly lost to the atmosphere during air-liquid contact. No further removal occurs during dual-media filtration.

A five-minute (empty bed) contact time carbon adsorber operating on dual media filter effluent maintained the concentration of the five organics to less than detectable levels for only 1-2 weeks. The order of breakthrough to 25 percent of influent concentration of the compounds assayed was, first, carbon tetrachloride, then trichloroethylene, dimethylphthalate, nitrobenzene and ethylbenzene. Only carbon tetrachloride and trichloroethylene had begun to break through the carbon at a contact time of 15 min after five weeks on stream. Complete loading data from the column operation, together with laboratory equilibrium tests will help to clarify the role of activated carbon in the removal of toxic organics from municipal wastewaters.

Many times the most economical way to increase the efficiency of a wastewater treatment process is to add onto or modify the treatment train. In that activated carbon is effective for removing many of the toxic compounds, a study has been funded to investigate the addition of powdered activated carbon to the aeration process in activated sludge. The grant was awarded to The University of Michigan for a three year study. Dr. Walter Weber will direct the research. Bench-scale complete-mix activated sludge units will be used to examine the mechanisms involved and the effects of operational variables on system performance. It is possible that only a slight amount of powdered activated carbon would be needed to remove toxic compounds when there are only trace amounts present in the influent.

In anticipation that process economics may be affected by the cost of powdered carbon, a study with Stanford University in California was initiated to seek a cheaper but equivalent powdered carbon to current commercial products. A variety of waste carbonaceous sources, other than coal or wood, are being treated by processes including pyrolysis and activation to produce a powdered activated carbon. The two major reactions will be optimized and the resulting carbons matched by performance against commercial products. The project is expected to be completed in March 1979.

The second approach to the problem of organic toxics is the determination of their treatability and biodegradability. Regulations cannot be made to control toxics unless the knowledge is obtained on how a compound behaves in the environment and in particular how it reacts in unit wastewater treatment processes. The task is tremendous because of the large number and variety of compounds now existing and the constant flow of new organic compounds. The task can be simplified if methods are developed to predict the behavior rather than having to resort to testing each compound under various environmental conditions. In the investigation of treatability, a variety of processes must be addressed, such as: adsorption (soils, solids and carbon), biodegradability (aerobic and anaerobic) volatility and chemical modification (by oxidation, precipitation, etc.). With enough basic information on the behavior of many compounds and their molecular characteristics, it may be possible to develop models which predict the behavior of untested compounds.

Another approach to this kind of research is to observe removals of compounds across various unit processes such as sedimentation, biological oxidation, carbon contactors, etc. Much can be learned from this approach about the unit process, but since many mechanisms operate in most processes, little fundamental knowledge will be gained about the compound itself. This research can be characterized as "removability." Because of the need for both short and long term information, research is taking both approaches of "treatability" and "removability."

The treatability of carcinogenic compounds is being conducted by Illinois Institute of Technology under a $69,000 EPA grant. The project will be finished this year, furnishing data on five compounds and their treatability by biological oxidation, activated carbon adsorption and chemical oxidation with ozone. Activated carbon adsorption capacities in mg of compound adsorbed per gram of carbon were measured. Values obtained for an initial concentration of 1.0 mg/l of test compound were: benzidine 170; β-naphthylamine 170; naphthalene 170; 4,4'-methylene-bis (2-chloroaniline) 720; and 1,1-dephenylhydrazine 160.

Chemical oxidation with ozone at 1.0 wt % and a flow of 2.4 l/min in a 12-liter reactor gave the following results: (a) an initial concentration of 10 mg/l of 1,2-diphenylhydrazine was reduced to zero in 6 minutes, (b) an initial concentration of 11 mg/l of β-naphthylamine was reduced to 0.01 mg/l in 4.5 minutes, (3) an initial concentration of 1.52 mg/l of 4-4'-methylene-bis (2-chloroaniline) was reduced to zero in 5.0 minutes, (d) an initial concentration of 7.4 mg/l of naphthalene was reduced to 1.5 mg/l in 40 minutes.

All of the compounds tested were amenable to biological oxidation. Activated carbon treatment was effective in removing all but the highly polar dimethylnitrosamine. Chemical oxidation was not effective for naphthalene or dimethylnitrosamine. A summary of the treatability data for the systems investigated is tabulated below. A "+" sign indicates effective removal of the compound by the treatment process while a "-" sign means little or no removal.

TABLE 5. TREATABILITY OF CARCINOGENIC COMPOUNDS

Compound	Biological Oxidation	O_3 Oxidation	Activated Carbon
Naphthalene	+	±(a)	+
1-1-Diphenyl Hydrazine	+	+	+
β-Naphthylamine	+	+	+
4-4'-Methylene-bis (2-chloroaniline)	+	+	+
Dimethylnitrosamine	+	-	-

(a) Reacts slowly with O_3. Some removal by gas-stripping.

Additional biodegradation studies are being done by Purdue University. They will study a number of representative priority organic pollutants in both aerobic and anaerobic experimental laboratory systems. It is hoped that after testing many compounds, the data will be such that one can predict the fate of organic compounds in biological wastewater treatment systems. Although the initial grant is for 2 years, it is expected that it will be continued so that Purdue could furnish quick response for organic compounds that in the future would be of great concern.

SLUDGE TOXICITY

Application of municipal sewage sludge on agricultural land is, in most cases, cost effective and ecologically sound. Sludge is a good conditioner and contains many essential plant nutrients; however, there is some serious concern about possible crop contamination by the toxic metals and organic compounds present in sludge. The heavy metal content of sludges, particularly that of cadmium, tends to inhibit the use of land for final disposal of sludge solids. The median values of cadmium and zinc for over 200 municipal sludges in the USA were found to be 16 and 1740 ppm, respectively [1]. For an industrial city, such as Chicago, the concentration of toxic metals is much higher. Typical values for Chicago are shown in Table 6.

TABLE 6. TOXIC METAL CONCENTRATIONS IN CHICAGO SLUDGE

Type of metal	Conc. in Dry Digested Sludge (ppm)
Cadmium	100
Chromium (total)	1000
Copper	900
Nickel	200
Lead	1800
Zinc	3500

Some toxic metals in sludge can be taken up by food crops after the sludge is applied to farm land. This contaminates the crops making them unsuitable for use.

There are two approaches to the problem of metal accumulation in sludge: (1) confine the metals at their source by pretreatment, (2) remove the metals from the sludge. Unfortunately to date, there is no practical way of doing either. Many of the metals come from diffuse sources, rather than from point discharge. The amino acids found in the microbial cellular proteins of secondary biological treatment processes form relatively stable complexes. Typical concentrations of amino acids found in activated sludge [2] and the stability constants with some metals [3] are shown in Table 7.

TABLE 7. AMINO ACIDS IN ACTIVATED SLUDGE AND STABILITY CONSTANTS WITH METALS

Amino acids	Concentration in act. sludge (% dry wt.)	Stability constant* of metal-amino acid complexes at 20-25 °C, 0.1 M ionic strength				
		Cd	Zn	Pb	Cu	Ni
Arginine	1.04 - 1.26	3.27	4.19	4.06	7.34	4.92
Cystine	0.18	-	9.86	12.2	-	10.48
Histidine	0.41 - 0.50	5.65	6.63	6.84	10.56	8.69
Isoleucine	0.91 - 2.20	-	-	-	-	-
Leucine	1.58 - 2.03	3.99	3.99	-	7.00	5.58
Lysine	0.92 - 1.33	-	(7.6)	-	(13.8)	(8.8)
Methionine	4.45 - 0.65	3.88	4.38	4.40	(14.75)	5.56
Phenylalanine	1.20 - 2.00	-	-	-	7.74	-
Threonine	1.15 - 2.20	-	-	-	{14.54}	-
Tryptophan	0.22 - 0.34	[8.1]	[9.3]	-	[15.9]	[10.2]
Valine	1.18 - 2.77	4.3	5.00	-	7.93	5.45
Tyrosine	0.70	-	{9.1}		{15.0}	{10.1}
Glycine	1.55 - 1.71	4.8	5.03	5.47	8.22	5.97
Glutamic Acid	2.89	4.78	5.45	-	7.85	5.9

*Symbols ( ): $[MA_2] / [M][A]^2$
[ ]: $[MA_2] / [MA][A]$
{ }: $[M(HA)_2] / [M][HA]^2$
Otherwise: $[MA] / [M][A]$

The technical and economic feasibility of removing heavy metals from sludge using a hot acid process has recently been studied under a contract with Walden Research, Abcor, Inc. The process involves acidification of the sludge (pH 2-3) and heating to temperatures below boiling (about 95 °C). Test results show that the process can solubilize the cadmium and the other heavy metals in varying percentages. In addition, the process improves the dewaterability of the sludge and destroys all pathogens. A preliminary economic analysis of the process indicates that it is cost-competitive with alternative stabilization and conditioning processes. A demonstration of the heavy metals removal processes at pilot scale is planned to confirm the laboratory findings and improve the predictability.

A sizeable portion of MERL's research effort has been devoted to determining the fate and effects of the nutrients and trace elements in sludge-treated soils. Copper, nickel and zinc have been identified as metals that are more likely to accumulate to phytotoxic levels in sludge-treated soils. Cadmium is the element of most concern because of its potential adverse effects on human health. It is assimilated by growing plants and is accumulated in the liver and kidneys of the consumer. Lead is of concern if sludge-contaminated forages are ingested by animals. Nitrates can leach into ground water, and both nitrates and phosphates can be transported by surface runoff into streams and reservoirs. Progress has been made in identifying acceptable sludge application rates and appropriate site management techniques for application of specific sludges on specific soil-climate-crop combinations. A grant to the City of Chicago is supporting a study of the accumulative effects of annual sludge applications over a long period of time. After 10 years, cadmium concentration in grain does not appear to be increasing. No phytotoxity from metals has been encountered. Corn yields from sludge-treated soils are equivalent to, or superior to, conventionally fertilized plots. Pheasants, rats and swine were fed corn from sludge-amended soils. Metals accumulated in livers and kidneys of pheasants, but not beyond the range of concentration in wild pheasants. In rats, there was a slight depression in weight gain of male rats. There were no adverse effects upon the swine.

With MERL support, the Science and Education Administration of the U.S. Department of Agriculture at St. Paul, Minnesota, is developing site management technology to enable the beneficial use of sludge as a soil amendment.

Conservation practices applied to a 16-hectare watershed include parallel terraces, grassed waterways, and a runoff detention reservoir. The parallel terraces divide the watershed into 13 test plots or subwatersheds, each of which is equipped with surface tile inlets to collect runoff water for analysis. Sludge is applied to subwatersheds growing corn at a rate of about 7.5 cm per year of liquid sludges containing 2 to 5 percent solids. Grass plots receive about 10 cm per year of the same sludge. Groundwater, soil water, runoff, soils, and plants are sampled and analyzed to determine the effects of using sludge. Two sludge storage lagoons are used. One is lined and the other is not. Sludge is analyzed for total and volatile solids, total and NH_4-N, conductivity, pH, organic matter, total P, K, Ca, Mg, and heavy metals. Water samples are analyzed for total N, P, and K, NH_4, and NO_3-N, PO_4-P, Na, Ca, Mg, conductivity, pH, and fecal coliform. Plants are analyzed for nutrients and metals, and soils are analyzed for the same parameters as the sludge.

Three and one-half hectares are used to produce corn for silage to be fed to lambs and dairy goats. Sixteen 0.22 hectare plots (4 replications and 1 control) will be treated with sludge at rates of 15, 30, and 45 tonnes per hectare per year. The control plot will be fertilized at a rate consistent with maximum corn yields.

The watershed study has involved application of 39 metric tons per hectare of sludge on corn and 43t/ha on grass. The sludge is low in cadmium (7 mg/kg) and has not resulted in an increase in Cd in corn. The unlined sludge lagoon was apparently effectively sealed as evidenced by lack of increased nutrients in soil water samples taken from under and below the lagoons.

Corn and grass yields were good on both sludge-treated and fertilized control areas in 1977. The greatest amount of surface runoff occurred during the snowmelt period. Nitrogen runoff was highest from sludge-treated corn, but runoff of phosphorus and potassium was highest on sludge-treated grass areas. Nitrates were increased in soil water, but could not be detected in groundwater. Lamb and dairy goat feeding of silage from sludge-enriched soils is underway.

Numerous efforts have been made to gain a better understanding of toxic hazards of modern synthetic organic chemicals that find their way into wastewater and sludge. However, there are many difficult sampling and analysis problems in concentrating, extracting, and identifying such compounds, many of which still may be unknown breakdown products of more complex chemicals that have undergone partial biodegradation. EPA is presently conducting a study to identify toxic materials in the effluent.

The utilization/disposal of sludge on croplands and pasturelands emphasizes the need for definitive data on plant uptake of synthetic organic chemicals. Uptake studies have shown that edible parts of plants contain these organics, but at levels 5 to 20 percent of the levels in the soils used. In general, root crops take up more chlorinated organics from the soil than other types of crops. However, studies have shown that chlordane, heptachlor and dieldrin are translocated from the soil into soybeans and stored in the oil of the seed [4]. Although the levels found were low, these data show that sludge can be a source of recycling of organic contaminants back into the food chain if sufficient precautions are not observed. An incident that happened in Bloomington, Indiana, a few years ago, illustrates this point. A cow was allowed to graze on pasture to which 27 tonnes/ha of sludge had been applied. It was discovered a few months later that the cow's milk contained 5 ppm of PCB's on a fat basis. Transfer of PCB's to the cow was probably related to grazing habits resulting in consumption of the contaminant without uptake of pasturage. Applying sludge to hay crops after cutting may not present a problem because much of the new growth takes place at the top, not from the bottom. It is not anticipated that organic toxics in sludge will be as great a problem as metals to land application of sludge.

SEWAGE SLUDGE PROCESS DEVELOPMENT

The sludge research program is divided into three areas: (1) sludge processing and treatment, (2) sludge conversion, and (3) beneficial utilization and disposal. Sludge processing and treatment includes all of the steps from the first appearance of sludge until it enters the disposal step. Anaerobic digestion is an example. Sludge conversion is any process that changes the nature of the sludge so that it is no longer recognized as sludge, for example composting. Beneficial utilization includes any constructive use of sludge. Land spreading of sludge on soil to add nutrients and improve the soil characteristics is an example of constructive use.

In recent years our sludge process development program has been switched from energy intensive processes, such as incineration, to obtaining useful products, such as compost, liquid and gaseous fuels. Our sludge management research program is continuously changing to be responsive to regulatory changes, needs of the wastewater treatment community and to shifting world situations.

THERMOPHILIC SLUDGE DIGESTION

The need for producing a stabilized sludge that is essentially free of pathogens is a controversial subject. In some disposal situations, such a need clearly exists. Thermophilic (ca. 50 °C) digestion processes can produce relatively pathogen free sludge.

Anaerobic and aerobic digestion processes are well established methods of stabilizing sludge, but both have certain inherent cost disadvantages. To achieve reasonable digestion rates, anaerobic digestion requires the addition of heat, and aerobic digestion requires an adequate supply of oxygen. By operating an aerobic digestion process in tandem with an anaerobic digestion process, these costs could conceivably be greatly reduced. If high purity oxygen is used for aerobic digestion, the temperature of the reactor can be raised to 60 °C, and at this temperature pathogenic organisms will be killed, and it will not be necessary to provide additional heat to the anaerobic digester. The methane produced can be used to meet other energy needs, offsetting the cost of oxygen. This concept is being tested by the City of Hagerstown, Maryland, on an EPA grant.

The Hagerstown, Maryland, wastewater treatment plant employs an activated sludge treatment process with a design flow of 30,000 m^3/d (8 mgd). The municipal utility in cooperation with the research personnel of the Union Carbide Corporation, Linde Division, will perform the study. The process was conceived by Union Carbide Corporation.

The anaerobic digestion process is a well-established solids handling system and used in sewage treatment plants throughout the country. It is, however, subject to upsets either due to overloading or buildup of toxic substances. On the other hand, the aerobic digestion process is more resistant to upset, but retention times for adequate stabilization are long, generally in excess of 20 days. In cold weather, the sludge temperature falls, and aeration for as long as 60 days is needed for adequate stabilization. Aerobic digestion can be made much more effective by using oxygen as the aeration medium instead of air. The operation can proceed at thermophilic temperatures (ca. 55 °C) without the need for supplementary fuel and is accomplished in less than 10 days. Oxygen costs, however, can be substantial.

The Union Carbide process takes advantage of the best features of both aerobic and anaerobic digestion. The rugged aerobic digestion process moderates the effect of shock loads and increases alkalinity, which stabilizes the subsequent anaerobic digestion step. No extra heat is needed for digestion and gas production is approximately the same, winter or summer. Because there is only one day of oxygen supply, cost of oxygen is low.

In operating the demonstration plant, the sludge in the aerobic digester will be treated with pure oxygen with a retention time of one day. The sludge will then be pumped into the anaerobic digester where it will be digested anaerobically for 8 days.

USE OF SOLAR ENERGY TO HEAT DIGESTERS

Digester gas is commonly used as fuel for digester heating. If solar energy could be substituted as the prime heat source for anaerobic digestion, then the methane produced could be freed for higher grade energy requirements. The technical and economic feasibility of providing this alternative heat source was evaluated by Environmental Systems, Incorporated, by the aid of an EPA contract.

The study showed that solar digester heating is economically feasible at all locations in the USA. The degree of economic attractiveness at any given location is directly proportional to the average annual solar radiation multiplied by the difference between the digester design operating temperature, 35 °C, and the average annual air temperature. The study, also, showed that optimum flat plate solar collectors can provide from 82 to 97 percent of the total annual digester heat load, the higher percentages being applicable to areas of higher solar radiation intensity. Specific guidelines have been developed for determining the optimum size and design of a solar heating system for any size of sludge digester at any location.

The town of Wilton, Maine, with the support of a construction grant from EPA and the State of Maine, is presently building a 1700 m^3/d (0.45 mgd) wastewater treatment plant that is designed to minimize the need of off-site energy. This project will demonstrate innovative energy savings concepts that will make use of an integrated energy system of solar energy, digester gas, and heat pump.

PARASITES IN SLUDGE

Public Law 92-500 emphasizes returning wastewater and sewage sludges to the land in an ecologically acceptable manner with minimal health risks. A 1977 Federal law prohibits ocean dumping of sewage sludge after 1981; therefore, more consideration is being given to land disposal. The potential for spreading of disease by poorly managed land application of sludge is real. The occurrence of bacteria and viruses in wastewater and sludges and their survival in the soil have been the subject of considerable research. Apparently, less interest has been stimulated for the occurrence of the larger pathogenic organisms. Few studies have includedthe surveillance of protozoa and helminths. This is a significant omission because some of these organisms can apparently survive wastewater treatment processes and land application better than any of the bacteria or viruses. Because of their weight, most cysts and eggs are found in the sludge.

A research grant was awarded to Tulane University to survey sludges from municipal wastewater treatment plants in the Southeastern USA. The first sampling has been completed. The first survey covered 38 municipal treatment plants. Of the 34 samples from drying beds sampled, 23 were positive for *Ascaris lumbricoides* (roundworm), 7 for *Trichuris trichiura* (whipworm) and 17 for *Toxocara spp.* The results indicated a much more general contamination of municipal sludges with *Ascaris* than had previously been expected. The study will be extended to other parts of the USA. Future studies will also include a mass balance study of parasites in conventional wastewater treatment systems and investigation of the effectiveness of parasite destruction by anaerobic and aerobic digestion and the effect of lime treatment.

As an adjunct to the survey for parasites, EPA has a grant with the College of Veterinary Medicine, University of Illinois, to study the transmission of helminth to livestock. In this investigation, anaerobically digested sludge containing ova of several nematodes, specifically *Ascaris sp.*, will be applied on strip mined soil which has not been contaminated by livestock. After variable quantities of sludge have been applied to plots, worm-free pigs will be placed on these experimental plots and will live in the natural environment with the ova-contaminated sludge. The rate of survival and transmission of parasitic helminths, and heavy metal accumulation in soil, feed, and swine tissues will be determined.

The first experiment with swine exposed on sludge-amended soils has been completed and parasites, heavy metals, and organic uptake are being measured. One swine from each pen (each pen contained 5 swine, 4 swine to be held for about 3 months) was necropsied after 3 weeks of exposure. The exposure in the pens was as follows: sludge and embryonated ova of *Ascaris lumbricoides suum*, embryonated ova of *A. lumbricoides suum*, and a control plot with no sludge or ova. A single larva was found in the lungs of the necropsied pig in the first and second case - the pens containing added embryonated ova. The second series of swine tests on sludge plots is underway.

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MEASURING THE DEGREE OF SLUDGE STABILITY

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ABSTRACT

There are several parameters that are applicable in the operation and control of sludge stabilization processes. Only a few have the potential of being useful in evaluating the degree of stability. The degree of stability required will depend on the method of ultimate disposal. Before a standard for determining the degree of stability of sludge can be established, more correlation studies are needed.

DEFINITION OF SLUDGE STABILITY

There are many definitions of "stable sludge." To most, stability is the antonym of putrescibility. To others, it means it can be applied to the soil without causing problems. The definition of stability implies the resistence to change. For sludge to be useful in land application, it is desirable that the sludge decomposes to a certain extent in the soil. Thus, ultimate stability is not desired, but rather a degree of stability in which the readily decomposable constituents have been degraded and only the more biologically resistant, decomposable compounds remain. The degree to which a sludge must be stabilized will depend on the location and the method of final disposition. A sludge can be stabilized by biological, chemical or physical means.

MEASUREMENT OF SLUDGE STABILITY

Several parameters have been used to measure the degree of sludge stability during stabilization processes. Thus, by taking a series of measurements during a sludge stabilization process, it is possible to determine when the parameter approaches a constant value, and the sludge becomes resistant to change. For example, reduction in volatile solids (VSS) could be used. Different control parameters could be used, depending on the unit process, e.g., aerobic digestion, anaerobic digestion, composting, etc.

It is much more difficult to determine if a sludge is stable, based only on terminal analyses. The control parameter VSS suggested above would have little meaning if only determined on the final product. A sludge with a high percentage of VSS is not necessarily unstable.

If standards are to be set for sludge stability, it is necessary to specify measurements and criteria that sludges must meet. For land spreading, aerobic activity would be the main consideration as an indicator of stability. While unsafe concentrations of metals, toxic compounds and pathogenic organisms are important in land spreading, they are not necessarily related to stabilization.

The literature is not uniform in discussing sludges. Should standards be based on only sludge solids or on the total sludge (liquid and solids)? The characteristics of the two are very different, especially in the case of anaerobic and aerobic.

The following are parameters that have been suggested as applicable to the measurement of the stability of sludges:

1. Aerobic Activity

The aerobic activity has been used as a method for calculating the BOD of a substance and for determining the respiration of an activated sludge. The value is usually given as mg of oxygen used by a gram of sludge per hour. High oxygen use per gram of solids would indicate a very active, viable sludge. A very low uptake rate would indicate a sludge that has been stabilized or that the sludge contains a toxic substance that inhibits the respiration of the sludge. Values between 0.5 and 1.0 mg O_2/g VSS·hr have been suggested for stabilized, aerobic digested sludge. The criteria used was that the sludge did not putrefy when left unaerated. The oxygen uptake rate of sludge will vary with temperature; therefore, the temperature of the test condition must be specified.

Oxygen uptake measurements offer a potential method of evaluating the degree of stability of aerobic digested sludges. More correlations are needed for different sludges, temperatures and ultimate disposal methods.

2. Reduction in Volatile Suspended Solids

In both aerobic and anaerobic digestion as the processes proceed, there is a drop in the percent VSS until a relatively constant value is reached. The reduction in volatile solids is a useful parameter for controlling the operation of digesters and will give an indirect indication of sludge stability if a series of measurements are taken. In general, aerobic and anaerobic digestion lowers the VSS by 50 percent. The reduction in VSS is a more reliable indicator in batch processes than continuous operations because of the periodic fluctuations in the volatile fraction of the sludge wasted to the digester. A complete mass balance would be necessary to determine accurately the sludge stability based on solids reduction. It is not a very useful parameter for determining the degree of stability of sludges when only the final product is analyzed. A sludge high in percent VSS is not necessarily unstable unless the solids are easily decomposed.

3. Odor Production

Odors created during the storage of sludge are of prime importance. Not only is the odor intensity important, but also the type of odor. Unfortunately, people disagree on what odors are offensive. What may smell unpleasant to one person may not be offensive to another. The most readily identifiable odor associated with sludges is that of hydrogen sulfide. Test procedures have been proposed for relating to the stability of sludges with production of hydrogen sulfide. A strip of lead acetate is hung in a bottle containing a sample of sludge. The time required for the paper to turn black is noted. The test period is generally 25 - 30 days. The long test period would limit its use for practical purposes, but it could be used to establish a process for stabilization. It would allow a direct comparison of aerobic digested processes, regardless of reaction conditions or feed sludge type. The environmental conditions of the test method would not necessarily reflect the conditions of actual field disposal. The test might be applicable to anaerobically digested sludge if modified. The hydrogen sulfide normally present in anaerobic digested sludge would first have to be destroyed or removed.

The Commission of the European Communities have defined a fully aerobic sludge as a sludge "where the Odor Intensity Index (OII) does not exceed 11 at any time prior to or during 14 days of storage at 20 °C unless the odor can be classified as a typical soil odor." Methods for determining Odor Intensity Index are given in "Standard Methods." The duration of the test does not make it a very practical parameter to use as a measure of sludge stability. Except for small treatment plants, the sludges are usually disposed of before 14 days. Although OII is an important parameter, for practical use some other parameter should be tied to OII.

OII probably would not be applicable to lime stabilized sludges. When lime is added to sludges, the odor usually changes from a rotten, offensive one to an ammonia or manure type and becomes more intense with time. Ammonia tends to disappear if the sludge is exposed to air.

4. Adenosine Triphosphate (ATP)

ATP is present in all living cells and has received some attention as a measurement of the biomass in the activated sludge process. In general, it is proportional to the mixed liquid suspended solids (MLSS). Unfortunately, wide variations can occur without significant changes in MLSS or COD in the system due to population shifts and changes in the phase of growth.

The test equipment is not available in many laboratories and requires a special firefly enzyme. The major drawback to the use of ATP for determining sludge stability is that the organic material need not be living to cause nuisances in the environment.

5. Total Organic Carbon and COD

It is possible to estimate the stability of a sludge by these parameters, but high values do not necessarily mean easily decomposable organic matter.

Some difficulty can be experienced in getting the sample comminuted so that it will not clog the total organic carbon analyzer.

6. Nitrification

Nitrification takes place during the latter stage of aerobic stabilization. The degree of nitrification is a fairly good indicator of the stability of aerobic stabilized sludge. Nitrification as a measure of stability would not be of any value for anaerobically digested sludge because the end product of the nitrogen cycle is ammonia.

7. Caloric Value

The fuel value of sludges is indirectly related to the VSS and TOC. There would be no advantage to using this parameter because VSS and TOC are more easily measured.

8. Other Suggested Parameters

For controlling the operation of an anaerobic digester, gas production can be used. Total carbohydrate, total protein, dewatering properties also have been suggested as suitable for describing the course of stabilization in an anaerobic digester. All of the values vary greatly depending on the type of sludge fed.

As a rough indicator for stabilization of both aerobic and anaerobic digested sludges, pH, specific resistance, BOD, orthophosphate and ammonia content of the sludge liquor can be used.

AIR POLLUTION FROM SEWAGE SLUDGE INCINERATORS: A PROGRESS REPORT

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ABSTRACT

Air pollution tests are being carried out at ten wastewater sludge incinerators to determine the quantity, particle size distribution, and composition of the particulate discharges. Results are now available for three incinerators.

The incinerators tested were equipped with low-intensity water scrubbers. Particulate discharges exceeded Federal standards for new sources. Percentage losses of lead and cadmium were much higher than losses of other metals. Lead and cadmium are concentrated in the fine particulate fraction leaving the incinerator. Since the scrubbers were inefficient in removing this fraction, concentrations of lead and cadmium were higher in the particulates leaving the scrubber than in those leaving the incinerator.

Maximum ground level concentrations of lead calculated from a plume dispersion model were approximately the same magnitude as local ambient concentrations. For cadmium, which has a much lower ambient concentration than lead, calculated maximum ground level concentrations were much higher than ambient concentrations. Use of high intensity scrubbers could reduce the local impact of lead discharges from sludge incinerators on ambient concentrations to a negligible value, but this may not be true for cadmium.

INTRODUCTION

In October of 1970, the Council on Environmental Quality recommended that ocean dumping of sludge be phased out as an ultimate disposal process for municipal sludge. With this recommendation in mind, the U. S. EPA undertook an investigation to determine whether this action would unreasonably limit sludge disposal options. A task force was set up in the Spring of 1971 to determine whether incineration, a potentially important disposal method for East Coast cities, was an environmentally acceptable disposal procedure. The task force investigated a number of sites where incineration was practiced and equipment was reasonably modern and working properly. Particulate discharge rates were measured, and particulate composition was determined. The task force concluded (1) that properly operated incinerators produced acceptable stack emissions of particulate matter, nitrogen oxides, sulfur oxides, and odor. On the basis of the task force work, standards for particulates were set.

The only EPA standards for sludge incineration are shown in Table 1 (2,3,4). No standards were set for oxides of nitrogen and sulfur, because quantities and concentrations were low compared to other sources.

TABLE 1: FEDERAL STANDARDS APPLICABLE

POLLUTANT	STANDARD
Particulates	0.65 g/kg dry sludge fed
Opacity	20%
Beryllium	10 g/24 hr.
Mercury	3200 g/24 hr.

The analytical determinations made on the particulate samples collected during the air pollution testing of the sludge incinerators indicated that there was a higher concentration of lead and possibly cadmium in the particulates than in the original sludge (on a combustibles-free basis) and the ash (1,5). Mercury was essentially all lost to the stack gases. Data obtained in a series of air pollution tests carried out in Japan (6) indicated that concentrations of some metals were markedly higher in the particulates collected from the gases leaving the gas cleaning equipment than in the original sludge (combustibles-free basis). A portion of these data has been recalculated and is presented in Table 2. The term C_p/C_s in the table is the ratio of the concentration of the metal in the particulates (C_p) to its concentration in the sludge on a volatiles-free basis (C_s). Examination of these ratios shows that cadmium and lead are greatly concentrated in the particulates, whereas iron and nickel show no concentration effect.

The cumulative effect of this information encouraged us to undertake additional studies of the air pollutants from incinerators. The objective of the investigation was primarily to determine whether the increased concentration of heavy metals in the particulates that was observed in Japan also occurred in sludge incinerators in the United States, and, if so, whether this condition might cause substantial environmental degradation. Tests have been inaugurated in two steps: four multiple hearth incinerators have been tested in the first step, and in the second step, six more incinerators, four multiple hearth and two fluid bed, will be tested. Results obtained on three of the first set of four incinerators tested are now available and are discussed in this presentation.

TABLE 2: METALS CONCENTRATION IN PARTICULATES RELATIVE TO FEED SLUDGE--JAPANESE INCINERATORS[1]

PLANT	TOTAL EMISSIONS (g/Nm^3)	METAL	CONCN. IN SLUDGE ($\mu g/g$)		CONCN. IN PARTICULATES		RATIO
			TOTAL	VOLATILES-FREE(C_s)	$\mu g/Nm^3$	$\mu g/g(C_p)$	C_p/C_s
K-1 (% ash = 27.5)	0.0295	Cd	18	65.4	160	5424	82.9
		Pb	335	1218	2700	91525	75.1
		Fe	40000	145500	140	4746	0.03
		Ni	70	254	40	1356	5.3
E-2 (% ash = 52.0)	0.1075	Cd	13.6	26.1	125	1163	44.6
		Pb	231	444	1095	10186	22.9
		Fe	24500	47115	3850	35814	0.76
		Ni	48.2	92.7	10	93.0	1.00

1. Calculated from data obtained from personal communication with Dr. A. Sugiki.

EXPERIMENTAL PROCEDURES

The four incinerators selected were multiple hearth furnaces of commercial size (ca. 20 ft. diameter) which receive sludges from municipal wastewater treatment plants with a high proportion of industrial waste. The stacks were sampled for particulates at the inlet to the particle collection device and at its outlet. Sludge, bottom ash, and water into and out of the scrubbers were sampled (all incinerators used wet scrubbers). It was desired to determine the particle size distribution of the particulates so a large sample was needed. This necessitated long sampling times. It was not possible to supply two sampling crews at each site, so the samples at the scrubber inlet and outlet were not taken simultaneously.

The sampling was accomplished using an Aerotherm* high volume Source Assessment Sampling System (SASS) train. The equipment is illustrated in Figure 1. In this system, the stack gas stream passes through a series of three centrifugal collectors followed by a filter, which segregates the particles into the following four fractions: greater than 10 μm, 10 to 3.5 μm, 3.5 to 1 μm, and 1.0 to 0.1 μm. For the sample at the scrubber inlet, where the quantities in each fraction were plentiful, sufficient material was collected to permit a further separation of the 10 to 3.5 μm fraction. This separation, which was carried out in a Bahco* classifier (7), divided the sample into a 10 to 5.6 μm and a 5.6 to 3.5 μm fraction.

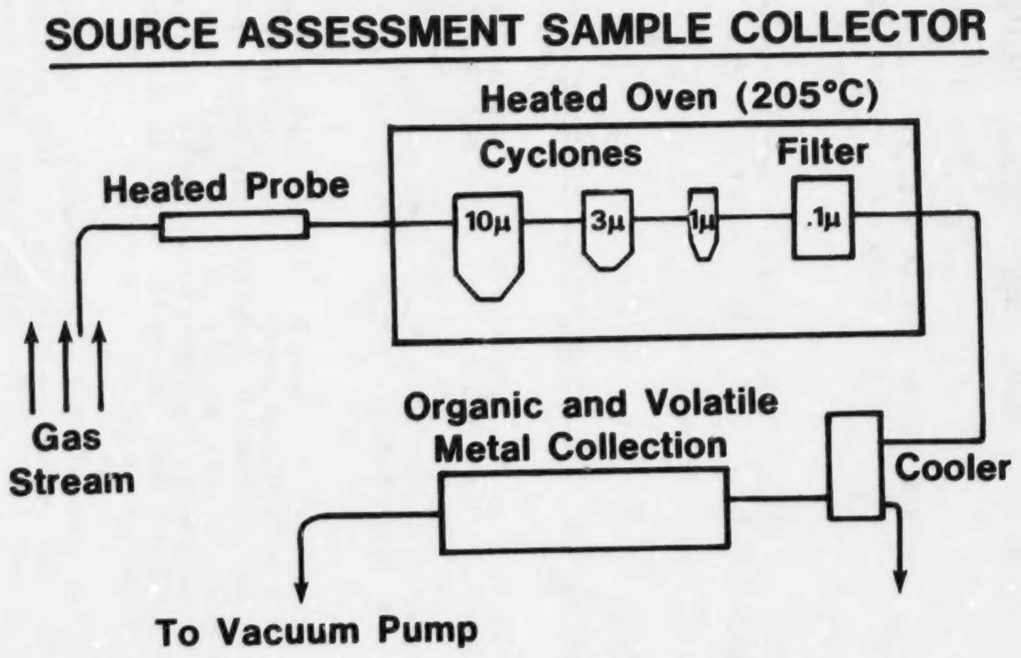
The SASS train is described in detail by Harris et al. (8). The section following the heated oven (see Figure 1) consists of a cooler and a series of impingers contained in an ice bath, where the stack gases can be reacted with adsorbents or liquid reagents to determine organic or inorganic gaseous substances. In the air pollution experiments conducted in this study, the impingers were filled with water. Analysis of the solids deposited in the impingers has not yet been completed.

Analyses for eleven metals were performed on each individual particle size range. Each fraction was digested and then trace metal analyses were performed using a Jarrell-Ash* spectrometer system with an inductively coupled argon plasma source for all of the elements except arsenic and mercury. These elements were analyzed by conventional atomic absorption. The results presented in this progress report are restricted to four metals: cadmium, lead, iron, and nickel. The important conclusions of the investigation are made clear with this abbreviated presentation. However, information on all of the metals will be presented when a comprehensive report of our investigation is published.

RESULTS AND DISCUSSION

The results of the stack testing are presented in Table 3. Although these incinerators were installed before Federal regulations went into effect, it is startling to note that none of them meet the Federal emission standard of 0.65 g/kg dry sludge solids for new stationary sources. As can be seen in the table, they all have low intensity scrubbers.

* Trade name. Use of trade names does not indicate EPA endorsement.



SOURCE ASSESSMENT SAMPLE COLLECTOR

FIGURE 1.

TABLE 3: RESULTS OF AIR POLLUTION STACK TESTS

PLANT	A		B		C	
TEST AT SCRUBBER	OUTLET	INLET	OUTLET	INLET	OUTLET	INLET
PARTICULATES						
Flow (kg/hr)	0.918	69.86	1.632	17.35	2.03	162.3
Conc. (g/dry Nm^3)	1.825	70.96	0.017	0.19	0.124	4.94
SLUDGE FEED						
Flow (dry, kg/hr)	1282	1736	2443	1999	1174	720
% Volatile Solids	55.5	42.2	50.2	51.7	40.6	29.4
SCRUBBER Δp(cm H_2O)	10.2	14.0	19.0	17.8	16.5	15.2
SOLIDS LOSS (g/kg DSS)	0.72		0.67		1.73	

Stack samples were taken at the outlet of the incinerator before the scrubber and at the outlet of the scrubber. The data permit calculation of a removal efficiency for the scrubber. It should be noted, however, that samples were not run concurrently but followed one another. Consequently, the efficiency calculation is approximate, although every effort was made to keep conditions constant.

The calculated efficiencies for Plant C and D are presented in Table 4. On the basis of the total particulates collected, the scrubber for Plant C is much less efficient than the Plant D scrubber. Examination of the efficiencies for each particle size fraction indicates much better agreement between the two units. Evidently, Incinerator C was underloaded relative to Incinerator D. Fewer particles were carried out of the incinerator, and they were smaller in size and harder to capture.

In the stack testing, rates of flow of solids entering the incinerator, and of particulates and gases before and after the scrubber were measured, and the solids collected were analyzed for the principal heavy metals. Consequently, the flow of solids at scrubber inlet and outlet can be related to the rate of sludge input. These results are presented for total particulates and four metals in Table 5. It is evident that the recovery of lead and cadmium is much poorer than for total particulates, iron, and nickel. Note that results appear inconsistent between scrubber inlet and outlet in some cases (for instance, in Test C only 7.2% of the total cadmium fed to the furnace escapes from the furnace and enters the scrubber, whereas 36% of the total fed to the furnace escapes from the scrubber). It should be recalled

TABLE 4: EFFICIENCY OF COLLECTOR FOR DIFFERENT SIZE FRACTIONS

PARTICLE SIZE RANGE (μm)	PERCENT REMOVAL PLANT C	PERCENT REMOVAL PLANT D
Total Sample	90.60	98.75
> 10	99.0	> 99.9
3.5 - 10	99.8	99.9
1 - 3.5	98.4	99.5
0.1 - 1	65.3	49.7
Inlet Loading (g/dNm^3)	0.192	4.94
Pressure Drop (cm H_2O)	17.8	15.2

TABLE 5: PERCENTAGE OF METALS FED THAT LEAVE IN THE PARTICULATE STREAM

COMPONENT	SCRUBBER INLET		SCRUBBER OUTLET	
	% REMAINING[1]	METAL/N.V.	% REMAINING[1]	METAL/N.V.
		TEST C		
Non-Volatiles (N.V.)	1.80	(1.0)	0.34	(1.0)
Cd	7.2	4.0	36	270
Fe	0.57	0.32	0.028	0.21
Ni	0.80	0.44	0.14	1.03
Pb	0.73	0.96	3.6	27
		TEST D		
Non-Volatiles	32	(1.0)	0.17	(1.0)
Cd	88	2.8	4.2	14.4
Fe	14	0.43	0.12	0.41
Ni	15	0.47	0.97	3.3
Pb	94	2.9	32	109
		TEST A		
Non-Volatiles	1.43	(1.0)	0.16	(1.0)
Cd	23	16.2	29	174
Fe	4.2	2.9	0.21	1.27
Ni	6.6	4.6	0.39	2.4
Pb	5.2	3.6	3.8	23.5

1. Percent of this component originally in the feed that is still not removed.

(see above) that the inlet and outlet samples were not taken at the same time. Composition of the sludge changed drastically halfway through Test C, evidently causing markedly different particulate compositions.

The results in Table 5 indicate the same kind of increase in concentration of lead and cadmium in the particulates when compared with the feed that was noted in the Japanese work (Table 2). It is perhaps more important to note that under some conditions over 30 percent of the lead and of the cadmium fed to the incinerator is discharged in the form of fine particulate matter even after wet scrubbing.

A part of the reason for the kind of effect noted in Table 5 is the relationship between particle size and composition in the particulates. The composition of cadmium, iron, nickel, and lead in the particulates entering the scrubber is shown as a fraction of particle size in Figures 2 and 3. Both cadmium and lead are concentrated in the finest fraction. When the particulates pass through the scrubber, the coarse particles are effectively removed, whereas the fine particles are not well removed. For iron and nickel, the composition in the various fractions is relatively uniform. Consequently, a concentration effect can occur for lead and cadmium, but not for iron and nickel, as a result of wet scrubbing. (Despite the concentration effect, total outflow of lead and cadmium is reduced by the scrubber.)

The selective removal of coarse particles in the scrubber is only part of the reason for the high proportion of cadmium and lead escaping from incineration. These metals and some of their salts are relatively volatile. Metal particles can vaporize, compounds can be reduced to pure metal in oxygen-deficient parts of the burning sludge and volatilized, and volatile salts can be vaporized. Oxidation of the metals probably occurs in the vapor phase, but the oxide particles are doubtlessly very small and remain entrained in the combustion gas stream, eventually to pass out of the incinerator.

The degree of contamination of the surrounding air from incinerator discharges can be calculated from plume models, which predict the ground level concentrations of particulates from a knowledge of the nature, quantity, and temperature of the discharge, and conditions of discharge such as velocity and stack height. The plume model used was supplied by EPA's Meteorology Laboratory, Research Triangle Park (8). The program assumes that the particles are of neutral buoyancy and essentially act like a gas.

The dispersion of a plume is illustrated schematically in Figure 4. The particle concentration is highest at the center of the plume and lower near the edges (Curves I-h in Figure 4). Curve I-L shows the longitudinal particle concentration distribution at ground level. The maximum concentration occurs some distance farther along after the plume first touches the ground. The plume dispersion model calculates the distance at which this maximum occurs and the value of the maximum concentration. The distance and the value of the maximum concentration shift with wind speed and atmospheric conditions. Plume dispersion models necessarily give approximate results because of the intrinsic difficulty of the problem as well as the fluctuations

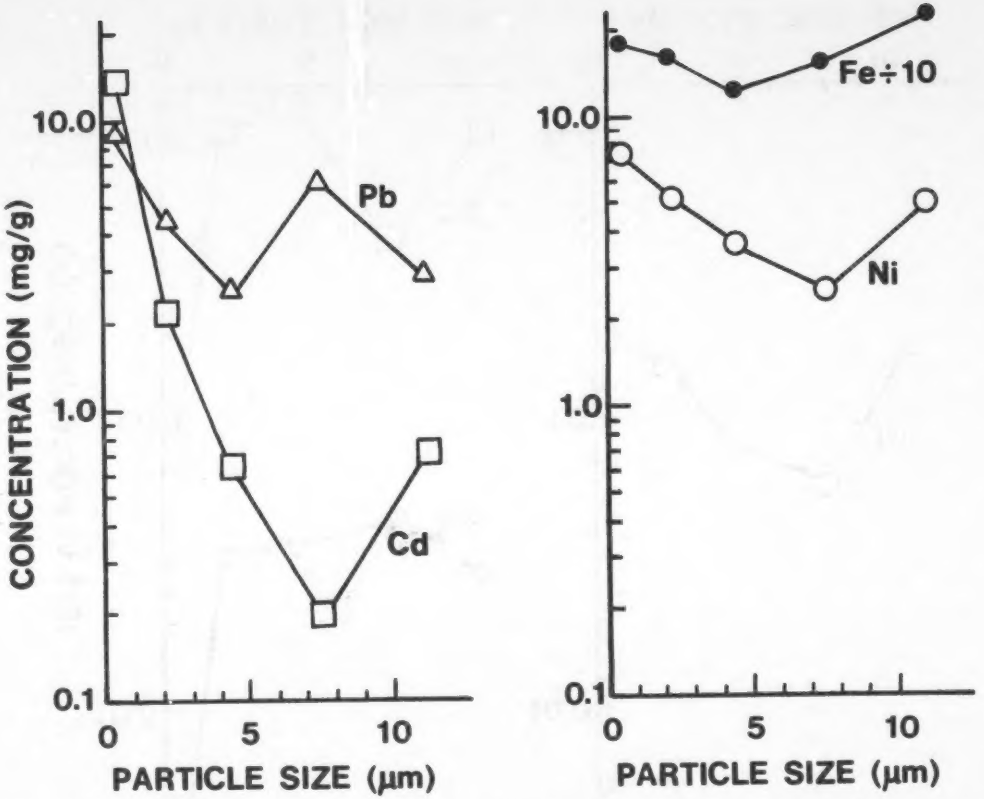


FIGURE 2. VARIATION IN CONCENTRATION OF METALS WITH PARTICLE SIZE: PLANT C

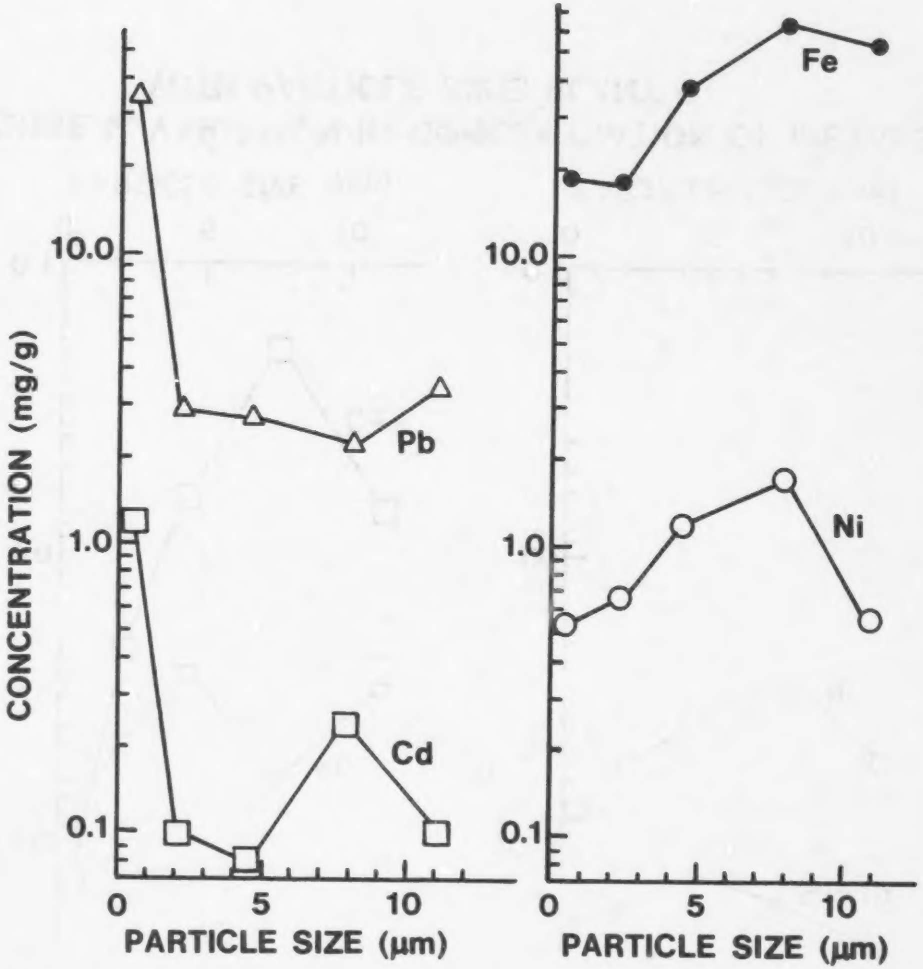


FIGURE 3. VARIATION IN CONCENTRATION OF METALS WITH PARTICLE SIZE: PLANT D

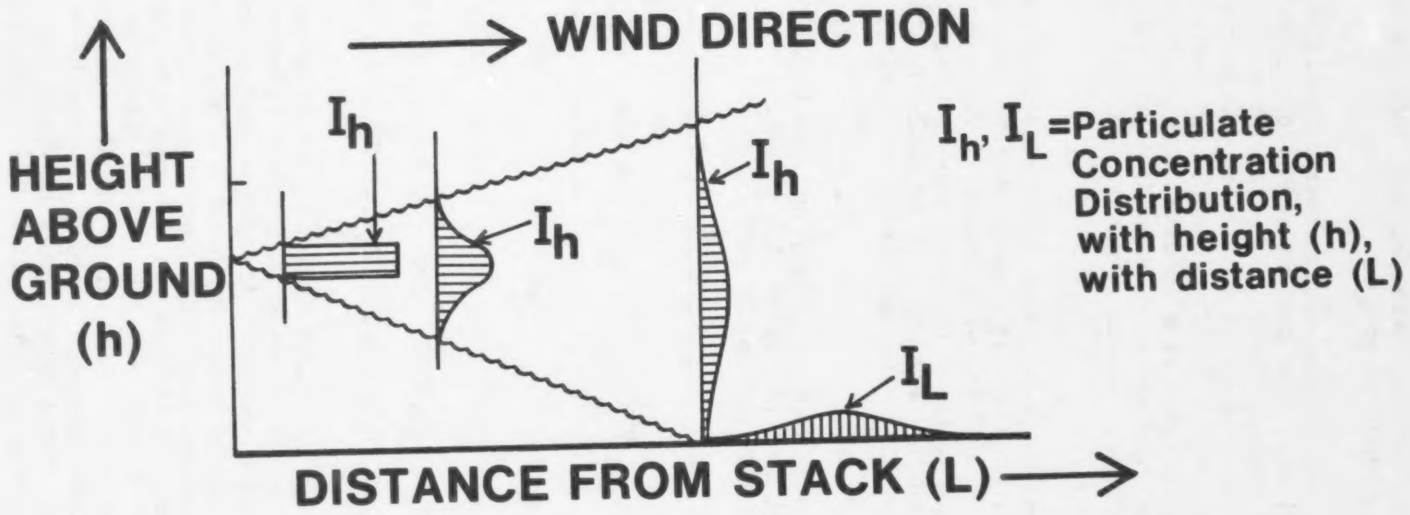

WIND DIRECTION
HEIGHT ABOVE GROUND (h)
I_h
I_h
I_h
I_L
I_h, I_L =Particulate Concentration Distribution, with height (h), with distance (L)
DISTANCE FROM STACK (L)

FIGURE 4. EFFECT OF PLUME DISPERSION ON PARTICULATE CONCENTRATIONS IN AIR AND AT GROUND LEVEL

of conditions that appear in nature. Nevertheless, they are useful in predicting the approximate magnitude of effects.

The results of these calculations are presented in Figure 5 for Plants A, C, and D for total particulates. The ambient conditions chosen were those that would contribute to a high ground level concentration--high solar radiation (a sunny day) and low wind velocity (0.5 meter per second was selected as a practical minimum velocity). The figure shows the maximum ground level concentration of total particulates as a function of wind velocity. The highest maximum ground level concentration depends on the site-specific conditions and, for the three cases, occurs at wind speeds that vary from 0.5 m/s to about 2 m/s.

In Table 6, the highest maximum ground level concentrations of lead and cadmium are shown along with the annual average ambient concentrations obtained in the cities where the plants were located. The ground level concentrations of lead are seen to be of the same order of magnitude as the ambient concentrations found in these cities. For cadmium, the ground level concentrations are far higher than the ambient concentrations in the cities.

TABLE 6: COMPARISON OF HIGHEST MAXIMUM GROUND LEVEL CONCENTRATIONS WITH AVERAGE CONCENTRATIONS IN TEST CITY

HIGHEST MAXIMUM CONCENTRATION ($\mu g/m^3$)	CITY A		CITY C		CITY D	
	PLANT A	AMBIENT	PLANT C	AMBIENT	PLANT C	AMBIENT
Total Particulates	6.24	--	8.31	--	15.4	--
Pb	0.59	1.71	0.33	0.65	0.70	1.50
Cd	0.085	0.0086	0.64	0.008	0.026	0.004

Estimation of the degree of hazard from a pollutant in an air-shed is an extremely complex process that requires site-specific information such as relative magnitude of regional and local discharges and meteorology. However, some general observations can be made. For lead, the highest maximum concentrations will under some conditions elevate ambient concentrations significantly, but these occasions will be rare and the area affected will be small. However, EPA has set a standard (9) for lead of 1.5 microgram per cubic meter average value. Many cities exceed this value. It is clear that there will be pressure to reduce lead discharges. There will probably be pressure to reduce lead discharges from sludge incinerators by using higher pressure drop scrubbers. This type of corrective action will very likely achieve the desired reductions.

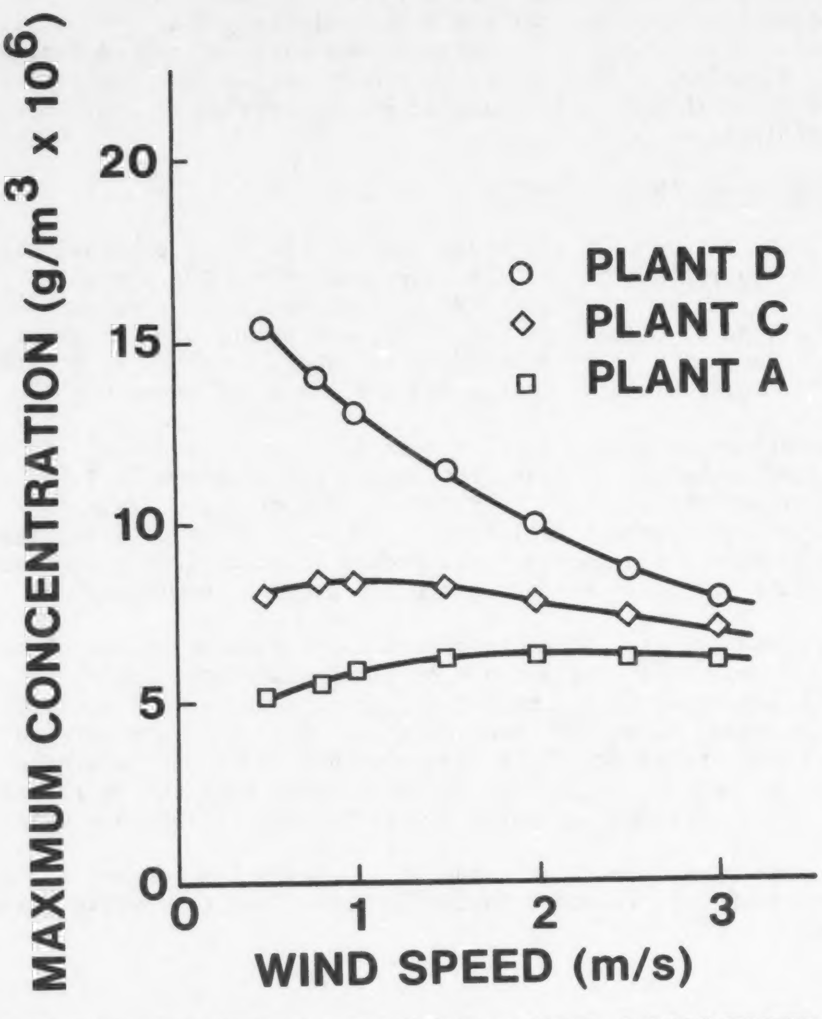


FIGURE 5. MAXIMUM PARTICULATE CONCENTRATION AS A FUNCTION OF WIND SPEED

For cadmium, the highest maximum concentrations greatly exceed ambient levels. No EPA standard for cadmium exists, so it is not possible to establish whether this situation is not in conformance with standards or is hazardous. However, with the serious attention being given to cadmium as a hazardous pollutant, these results will receive prompt attention. The pressure to improve performance of scrubbers will be substantial. Use of higher pressure drop wet scrubbers may not reduce discharges to a point that ground level concentrations from incinerator plumes will not exceed ambient concentrations. A radical change in the method of particulate removal or a change in the method of thermal processing from incineration to starved-air combustion or pyrolysis may be needed.

CONCLUSIONS AND RECOMMENDATIONS

The low pressure drop scrubbers used at the sites examined allow particulate concentrations in the discharges to exceed EPA standards for new stationary sources. In addition, under some conditions, as much as 30 percent of the lead or cadmium in the sludge can escape as fine particulate matter. The use of high pressure drop scrubbers is strongly recommended to reduce total particulate discharge and the losses of heavy metals.

Calculations of ground level concentrations of lead indicate that occasionally discharges of lead from sludge incinerators will noticeably increase local ambient lead concentrations. Use of higher intensity wet scrubbers would probably make this likelihood remote. Cadmium discharges from sludge incinerators can noticeably increase local ambient concentrations. High intensity scrubbers may not solve the problem completely.

The method used to calculate ground level concentrations of cadmium and lead gives "worst case" estimates. An alternative approach is to calculate the maximum annual average ground level encountered in the vicinity of the incinerator, based on the emission data and available data on wind velocity and atmospheric stability. This figure should be better related to long term impact on people living near an incinerator than the "worst case" estimate. This alternative approach will be used in future presentations.

More experimental work is needed to identify the reasons why lead and cadmium are so poorly retained in incinerators, and to develop means to retain them.

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PROGRESS IN INSTRUMENTATION AND AUTOMATION

by

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Introduction

One of the primary goals of U.S. EPA research and development activities is to devise methods to improve control of treatment processes and systems. Improved process control will result in cost reduction and discharge of effluents with lower pollution potential. One of the primary methods of achieving improved process control is through the application of instrumentation and automation. Instrumentation and automation provides for much closer monitoring of treatment systems and allows for much more rapid response to upsets than can generally be achieved with manual operation.

In order to tap the potential advantages of this area of technology the U.S. EPA has for several years sponsored a program of research and development on instrumentation and automation for control of wastewater treatment systems. A review of the activities of this program was given at the Fourth United States/Japan Conference on Sewage Treatment Technology (1). Some of the more recent studies will be covered in this presentation. The information presented will be devided into several distinct areas including: instrumentation testing, process control strategy development, and digital computer applications to plant and areawide control.

I. Instrumentation Development and Testing

A recent survey by EPA of performance and acceptance of instrumentation at wastewater treatment plants identified numerous instances of unacceptably high failure rates and unsatisfactory operation (2). Among the several contributing factors identified were the absence of proven performance criteria and technical specifications for the instrumentation, and accepted methods for field maintenance and testing of instruments.

a) Testing and Certification Laboratories

In order to remedy this situation on Inter-Agency Agreement was established with the National Bureau of Standards to develop performance criteria, specifications and test methodologies of on-line instrumentation for use in wastewater treatment. The project will deal with fluid flow instruments and chemical instrumentation. Table 1 lists the instrumentation which will be investigated.

For each instrument or group of instruments three levels of reports will be produced. The first level will deal with the performance specifications of the instrument as a function of its intended use, the essential procedures for maintenance of the equipment in a wastewater treatment plant, and factors required to maintain compatibility of the instrument with other electronic or pneumatic components of automatic process control systems. These data will generally be available from vendors literature, ISA and ASTM specifications and knowledge of the level of confidence in a measurement needed to achieve control in a wastewater treatment plant.

TABLE 1

INSTRUMENTATION INVESTIGATED IN NBS STUDY

Flow

Venturi Type Meters
Magnetic Flow Meters
Acoustic Flow Meters
Flumes

Chemical

Residual Chlorine
Dissolved Oxygen
Total Organic Carbon
Suspended Solids
pH

The second level report will deal primarily with development of a "Test Protocol" i.e. methods of testing to determine if the performance specifications for an instrument are being met. The test protocol will include: installation procedures, test procedure to generate baseline data, calibration procedures, maintenance during test, and other operational guides.

For the most part the "test protocol" described above will address bench testing of the instrument. A test protocol referred to as a Measurement Assurance Program dealing with field evaluation and calibration procedures will be the subject of the third level report.

The first level reports were submitted within the last month. The second and third level reports will not be complete for 1 to 2 more years. Extensive bench and field testing will be necessary to prove out the test protocols.

A second effort in this area, which has just been started, is a Feasibility Study on Establishing an Instrumentation Testing and Certification Institute. In this project which has been undertaken by Public Technology Inc. a study will be made of the legal, institutional, financial, technological, and market factors involved in establishment of such an independent institute (3). Particular attention will be paid to the experiences of similar types of institutions in the United Kingdom (Sira Inc.) and the Netherlands (W.I.B.). These institutes conduct tests at the request of manufacturers or potential users of instrumentation to determine if an industry wide or manufacturers specification can be met by the instrument. All reports on results are confidential. It is expected that the results of the NBS study will form the base testing program for use by an type of testing and certification center.

b) Field Evaluations of Instruments

Recently evaluation of several on-line analysers was conducted on EPA projects. These included a TOC analyser used in conjunction with an F/M control strategy for activated sludge systems, and a sludge blanket level monitor and a solids concentration monitor used in conjunction with control of a thickener.

The EPA instrumentation and automation program has had only limited experience in making first hand evaluations of instrumentation, because basic policy is to refrain from conducting specific instrument evaluation under government auspices. However in the course of conducting evaluations of specific control strategies of treatment processes evaluations of some equipment are conducted. These evaluation are in the context of the control strategy being evaluated i.e. the performance of the instrument must be good enough so that the automated control strategy can be properly implemented. Thus, even if an instrument performs its basic sensor function properly it may not be acceptable if its response time is inadequate to the control strategy.

The TOC analyser evaluated was a Dohrmann/Envirotech, Model DC-60. The operating principles of the DC-60 are analogous to laboratory techniques for the determination of organic carbon as described in Standard Methods, 13th edition, except the sample is not homogenized, and the carbon content is equated to the area under the peak rather than the amplitude of the peak(4).

As shown in the system plumbing diagram, Figure 1, a sample of the stream to be monitored is diverted to a small, (500 ml), overflow tank adjacent to the analyzer. The sample is pumped from the overflow tank at 25 ml/min to a baffled chamber where it is continuously acidified to a pH less than 2 and air sparged to remove inorganic carbon. The conditioned sample is pumped at 20 ml/min past an automatic syringe (injector valve) to drain. At a selectable interval, 5, 10, 15, 30, 60, 120 minutes, a 250 micro-liter portion is drawn from the 20 ml/min sample stream and injected into the top of a combustion chamber, Figure 2. Here the 250 micro-liter sample is vaporized at 900°C in an air stream. Under the influence of a catalytic heat transfer medium, (cobalt oxide) carbon compounds are converted to carbon dioxide. The resultant sample is cleaned of non-gaseous material and passed through an infrared detector. The carbon dioxide content of the sample gas is measured by comparing the infrared light passed through the sample gas to the infrared light passed through a reference cell.

The resulting electrical signal is amplified, integrated, and displayed on a strip chart recorder. The recorder is calibrated to read directly in milligrams per liter of total organic carbon. An electronic sample sensing device automatically shuts down all three pumps upon loss of sample flow, and furnace temperature runaway is prevented by a thermostatic power cut-off. An adjustable high level alarm signals when TOC exceeds a selected level, a predetermined number of times in succession. Connections are available at the rear of the analyzer for remote monitoring of; the high TOC level alarm signal, the sample loss alarm signal, the furnace overtemperature alarm signal, and an isolated output signal proportional to the TOC value.

During a 1 month continuous evaluation at the Blue Plains Pilot Plant the TOC analyser sampled and analysed primary effluent once every 5 minutes. During this period TOC varied from 30 to 100 mg/l. Comparison of the analyser results to samples analysed manually on a Beckman Model 915 TOC analyser indicated a maximum deviation of ± 2.5 mg/l TOC. A similar successful evaluation for use on tertiary effluent in the TOC range of 0 mg/l to 10 mg/l was conducted. Here the maximum deviation was ± 0.5 mg/l. The only difficulty encountered was the need to recalibrate the instrument span once during each evaluation.

In order to use the analyser out-put to implement F/M control strategies it was necessary to introduce a smoothing routine. The analyser tracked small changes in influent TOC so closely that direct use of the out-put would have introduced oscillations in the activated sludge reactor solids. An inexpensive microprocessor can be used to automatically implement the smoothing routine.

The sludge blanket level detector (Biospherics Model 56) was installed in a thickener at the MWCC of Minneapolis-St. Paul Metro Plant. The unit, consists of two submersible probes "A" and "B" suspended, by individual

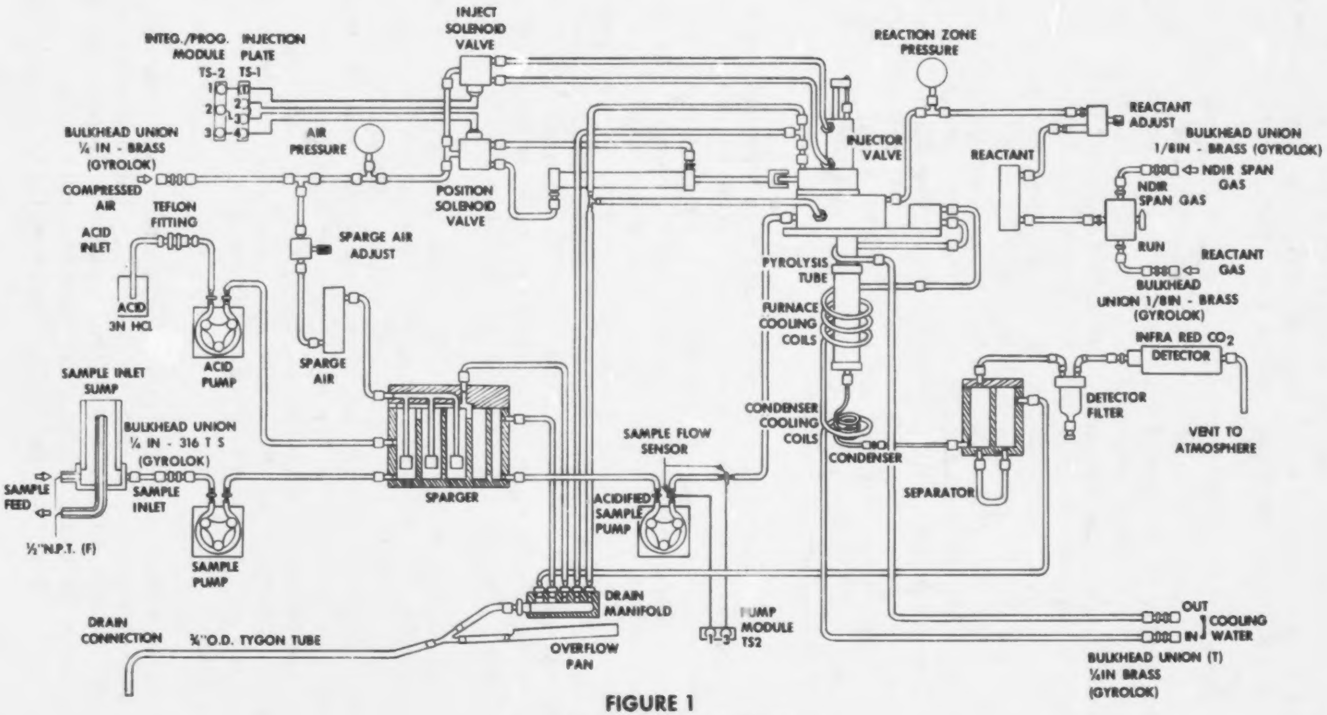
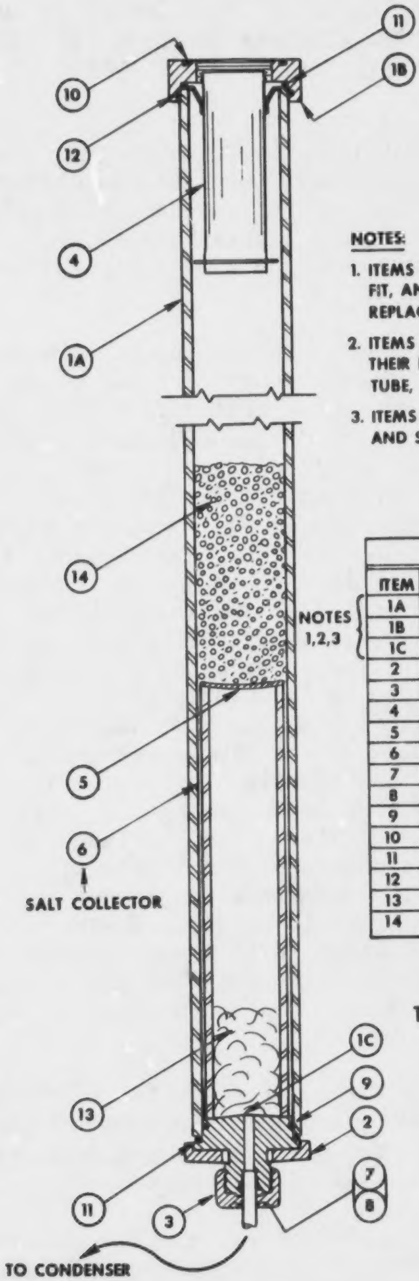


FIGURE 1
TOC ANALYSER PLUMBING



NOTES:

1. ITEMS 1A, 1B AND 1C ARE ALL CUSTOM FIT, AND ARE NOT INDIVIDUALLY REPLACEABLE (NOTE 3).
2. ITEMS 1B AND 1C ARE MATCHED TO THEIR RESPECTIVE ENDS OF THE REACTION TUBE, AND MAY NOT BE INTERCHANGED.
3. ITEMS 1A, 1B 1C, 2 AND 3, ARE AVAILABLE AND STOCKED UNDER PART NO. 835-275.

BILL OF MATERIAL		
ITEM	PART NO.	DESCRIPTION
1A	030-400	REACTION TUBE
1B	835-281	CUSTOM TOP TUBE NUT
1C	835-282	CUSTOM BOTTOM END PLUG
2	835-244	END PLUG JACKET
3	835-285	END PLUG NUT
4	835-276	HASTELLOY LINER
5	835-284	PERFORATED CERAMIC DISC
6	030-402	CERAMIC INSERT
7	080-834	TEFLON FRONT FERRULE
8	080-836	STAINLESS BACK FERRULE
9	100-472	O-RING -028
10	100-473	O-RING -029
11	100-474	O-RING -030
12	100-475	O-RING -031
13	511-735	QUARTZ WOOL
14	511-870	OXIDIZER GRANULES

FIGURE 2
TOC ANALYSER COMBUSTION CHAMBER

waterproof cables, from a control box. Each probe consists of a light source and photocell separated by a 3/4 in. sensing gap. The light transmitted to the photocell is a function of the concentration and characteristics of the solids slurry in the sensing gap. When in service each sensor is suspended in the thickener from its cable which also provides power to the light emitting diodes and photocell and transmits the photocell output to the control unit.

The control unit is housed in a NEMA 4 enclosure mounted on the thickener handrail and consists of control circuits, power supplies and output relays. A two position response switch is provided for fast or delayed response to changes in photocell output. Three indicating lights labeled, LOW, MEDIUM, HIGH, are mounted on the enclosure to indicate the sludge blanket position as below, between or above the two sensors.

When probe "A" is located above probe "B" in the thickener basin the control unit functions as follows. The energy source in each probe emits radiation through the optical sensing gap to the photocell which senses the amount of energy transmitted through the liquid and produces a signal related to the suspended solids concentration of the liquid in the gap. This signal is compared in the control circuit to an adjustable density threshold setting. When the signal representing the concentration of solids in the liquid at probe "A" exceeds the threshold setting an electronic 15 min timer is activated. If the sludge blanket level remains at or above probe "A" at the expiration of the 15 min interval the output relay is energized and the HIGH lamp on the control box illuminates. The output relay is used to energize the starter of the thickener underflow pump motor. After a period of sludge withdrawl the sludge blanket level drops below probe "A" and the photocell output drops below the threshold setting. At this time the HIGH lamp is extinguished and the MEDIUM lamp illuminates. After continued sludge withdrawl the sludge blanket drops to the level of probe "B" and the signal representing the concentration of solids falls below the threshold setting. At this point in time a second 15 min interval timer is activated. If the sludge blanket level remains below probe "B" at the expiration of the 15 min interval the output relay is deenergized and the MEDIUM lamp is extinguished and the LOW lamp illuminated. The underflow pump starter is disengaged when the output relay is deenergized. When the sludge blanket level rises to probe "B" the photocell output increases above the threshold setting and the LOW lamp is extinguished and the MEDIUM lamp illuminated. The above cycle is repeated as the sludge blanket level rises to probe "A".

A three position switch (HAND, OFF, AUTO) was installed in the control room to allow the operator to establish the mode of operation. This flexibility was required to minimize problems associated with instrument failures and high sludge levels in the holding tanks downstream of the thickeners.

An evaluation of the efficacy of this device was made in terms of blanket level stability versus an identical thickener receiving identical feed. In the control thickener periodic manual measurements were used to determine blanket depth, and underflow pumps were started or stopped manually from a control center to keep the blanket at the target depth. In the automated thickener a signal from the blanket level instrument controlled the pumping.

Figure 3 illustrates results obtained over a 10 day period. It can be seen that the unit with the automatic blanket level detector had a much more stable blanket and that the blanket was kept within 1 foot of the set point. Deviation from the set point was due to fouling of the sensor. It was found necessary to clean the probe once per week to avoid problems.

Optical type solids analysers were installed in the inflow, overflow, and underflow streams of these thickeners to aid in the collection of data on thickener performance and to more fully evaluate their own performance characteristics. Biospherics model 52LE suspended solids analysers and Biospherics model 52H sludge density analysers were utilized. These are self-cleaning optical types of analysers.

The analysers were installed in the influent and underflow pipes through a pipe insertion adapter mounted in a 2 in. NPT threaded hole. The overflow analysers were mounted in the sample sinks in the thickener gallery. The installation locations are illustrated in Figure 4.

The influent solids analysers consisted of a one foot long sensing head housing a glass sampling chamber, a motor drive mechanism that operated a plunger positioned in the sampling chamber, and a control unit connected to the sensing head by a multi-conductor cable. The sensing head of the analyser contains a light source that transmits a light beam across the sample chamber to a photocell. The plunger is equipped with a wiping seal that cleans the optical surfaces of the photocell and light source with each operating stroke. The signal from the photocell is linearized in the control unit producing a meter reading that is designed to be proportional to the suspended solids concentration of the sample.

The unit completes a sample and analysis cycle every 15 sec. The plunger retracts and draws a sample into the sampling chamber similar to the operation of a common laboratory syringe. The light transmission measurement is made and the sample is expelled. The instrument output (meter and/or recorder) is maintained constant during each cycle. A range switch on the face of the control unit provides for operation in the range of 0 to 3000, 0 to 10,000 and 0 to 30,000 mg/l. An ON/DAMP switch allows for recording of the actual output fluctuations at 15 sec intervals or only a portion of the step change when the fluctuations are large.

Zero and span controls are located at the rear of the control unit. The zero control is used to adjust the output with clear water present in the sampling chamber. The span control is used to adjust the slope of the calibration curve. A test plug which consists of two fixed resistors and a high/low switch can be attached to the control unit in place of the signal cable. The resistors simulate photocell output in the 0 to 10,000 and 0 to 30,000 mg/l ranges; and, the meter readings with the test plug in place can thus be used to identify malfunctions in both the control unit and sensing head.

The overflow analysers are functionally identical to those used to monitor inflow, however, the sensing heads are four feet long. The units were mounted in sample sinks in the thickener basins. Thickener overflow

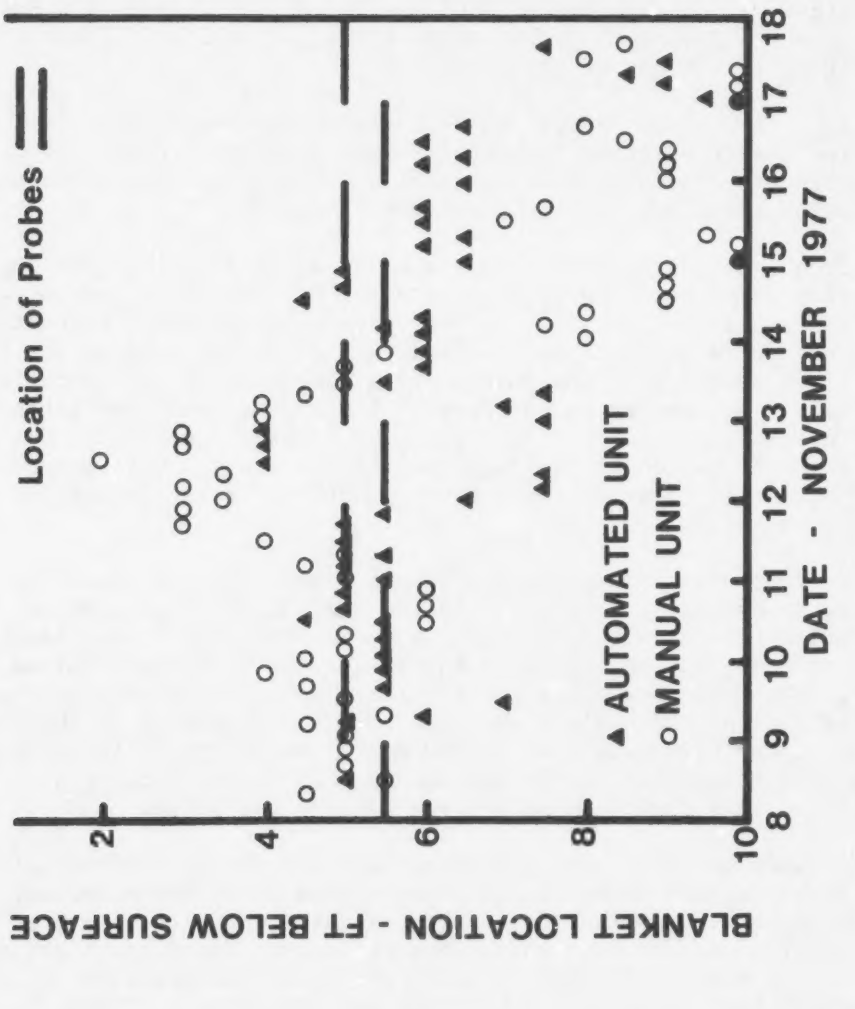


FIGURE 3 COMPARISON OF BLANKET LEVEL CONTROL

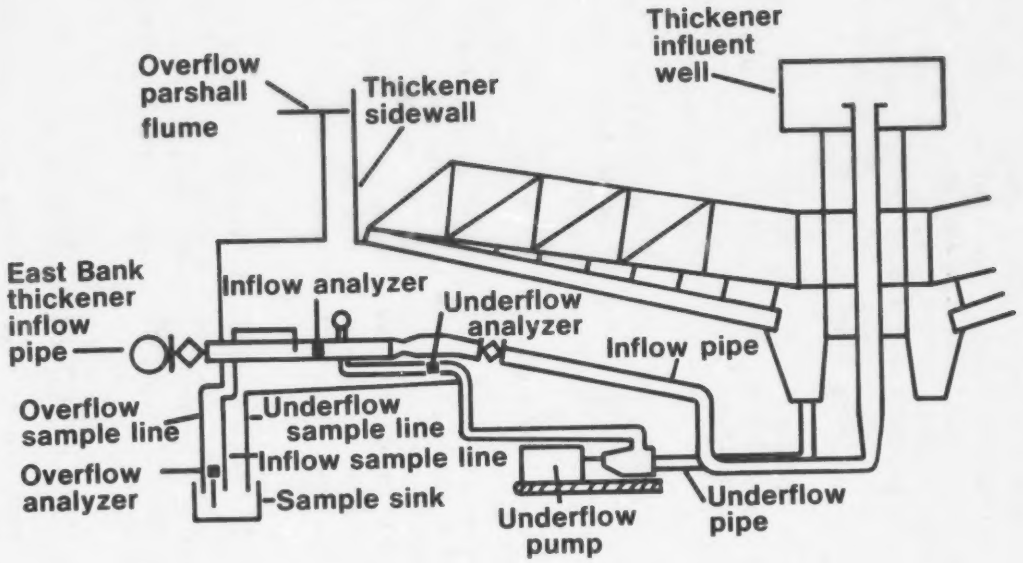


FIGURE 4 LOCATION OF SOLIDS ANALYZERS AND SAMPLING LINES

is piped from the overflow flumes to the sample sinks and discharges to sample buckets. The analysers are supported above the sample sinks and the sensing heads extend into the sample buckets which overflow continuously.

The sensing head of the underflow monitor is externally identical to the inflow unit, however, the underflow analysers contain three photocells in the sensing head; one each for measuring light transmittance, 90° light scatter and color compensation. The parallel combination of the light transmittance and light scatter photocells provides an output related to the suspended solids concentration in the sludge sample. The color compensation photocell adjusts the input current to the control unit amplifier to reflect changes in sludge color. A linerization module in the control unit corrects the nonlinear response of the transmittance and light scatter photocells. The unit completes a sample analysis cycle every 40 sec.

The control units are equipped with the ON/DAMP switches and span adjustment controls. The instrument range is fixed at 0 to 10% solids and is calibrated in terms of total solids. A test plug is used to simulate the output of the sensing head and check for drift in the control circuitry.

The outputs of the six solids analysers along with the output of the thickened sludge (underflow) flowmeter serving basin 4 were recorded on a Leeds and Northrup Speedomax W multipoint recorder.

The performance of the solids analysers was monitored during a 4 month period while they were on-line. The inflow and overflow sensors performed quite well. The calibration did drift slightly for each of the devices but the correlation coefficient between the instrument and manual analysis remained high. For one of the inflow sensors it decreased from 0.998 to .967 and for the other from 0.973 to 0.952 over a range of suspended solids of 0 to 8,000 mg/l. For one of the overflow sensors the correlation coefficient decreased from .997 to .991 and for the other from .998 to .967 over a range of 0 to 6,000 mg/l.

For the underflow solids analyses generally poor results were obtained These were found to be due to changes in the physical properties and color of the sludge rather than any electronic defects in the sensor. A test with this same instrument for a limited time about 1 year prior to this test had given good correlation ($r = 0.90$).

II. Treatment Process Control Strategy Development

During the past few years EPA research efforts have concentrated on development of new or verification of existing control strategies for the activated sludge process and selected sludge handling and stabilization techniques. Investigation of these was deemed most important because of the large expenditures scheduled for these processes under the construction grant regulations of PL 92-500. In conducting these studies emphasis was placed on demonstration of the control strategy concept, and deliniation of improvements in process performance and or process cost effectiveness.

a) Activated Sludge

For the activated sludge process emphasis has been placed on strategies involving dissolved oxygen level control, and instantaneous food to microorganisms ratio control (F/M). The need to maintain a minimum level of dissolved oxygen in the aerator of an activated sludge system was established many years ago, and the potential power savings when this is automated is self evident. Consequently the major recent effort was to produce a design hand book which could be used to aid engineers in designing the most cost effective systems. This manual which was published in 1976 discusses the various activated sludge modifications, aeration methods, equipment and application techniques, compressors and blowers, D.O. control methods, and presents an economic analysis of manual versus automatic D.O. control. The reader is referred to the manual for the findings (5). In addition detailed case histories were presented for 12 sites at which both manual and automatic D.O. control were conducted. Table 2 presents a summary of the improvements resulting from the use of automatic D.O. control.

The value of instantaneous F/M for control of activated sludge has been a subject of great debate in the environmental engineering field for a long time. One problem is that F/M control is often confused with Solids Retention Time (SRT) control. The latter has proven to be a valuable control parameter Confusion results because the long term average F/M in an activated sludge is a function of the SRT; indeed these parameters are mathematically related by a simple equation. Thus various investigators have interchangeably used these two terms. Instantaneous F/M, however, can vary significantly over limited time periods while SRT remains constant.

An investigation of the efficacy of instantaneous F/M control was conducted in a joint EPA-Clemson University study (6). The Clemson University staff prepared a model of the activated sludge system and used this model to simulate events which would occur if an activated sludge system was subjected to a typical diurnal variation of loading, and an attempt was made to maintain instantaneous F/M constant. It was found that this was impossible in a conventional type activated sludge system (Figure 5a); because the clarifier can not store sufficient solids to effect good control while still maintaining good solids-liquid separation. In addition when increased recycle is used to try to increase the mass rate of return sludge the sludge concentration in the clarifier and recycle is diluted so that the mass rate of return does not change. Use of a separate sludge storage basin between the clarifier and aerator (Figure 5b) was found necessary to achieve instantaneous F/M control. The mathematical simulation indicated that a storage basin with a volume equal to 40% of the aerator volume was sufficient to achieve good instantaneous F/M control. Use of such a storage chamber essentially converts the flow sheet to the contact stabilization mode of operation.

An attempt to obtain experimental verification of this model and determine the specific effect of instantaneous F/M on activated sludge performance was initiated at the Blue Plains Pilot Plant. Unfortunately the work had to be terminated before all the planned runs were complete. Table 3 summarizes a comparison between a run with no control of instantaneous

TABLE 2

EFFECT OF D.O. CONTROL ON ACTIVATED SLUDGE

Parameter	% Improvement
Air Supplied per Unit of BOD Inflow	21.9
Air Supplied per Unit of Inflow Volume	11.6
BOD Removed per Blower KWH	32.1
BOD Removal Efficiency	3.2

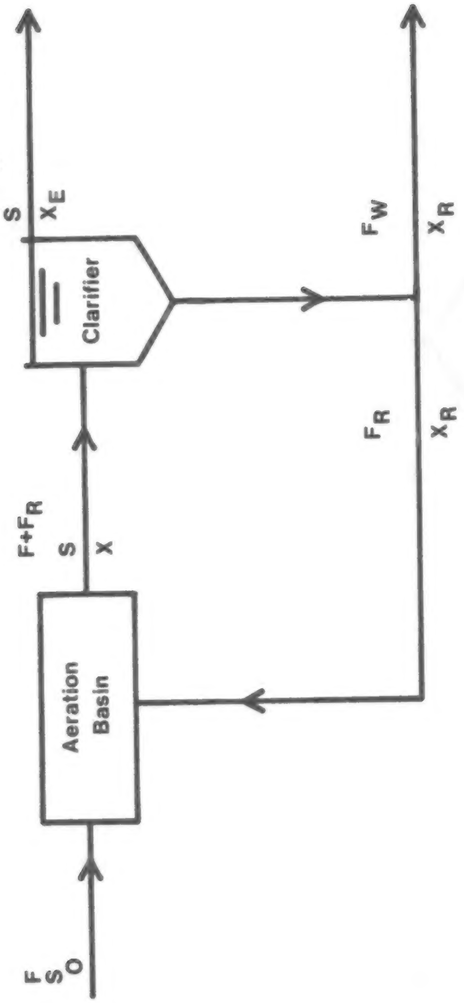


FIGURE 5A CONVENTIONAL ACTIVATED SLUDGE

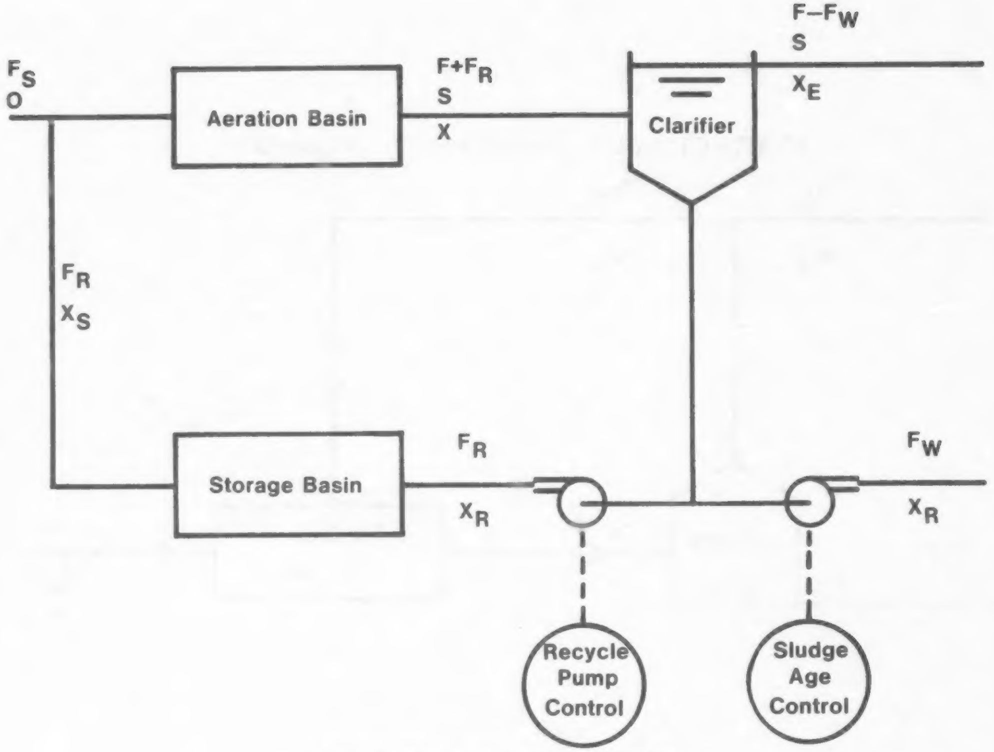


FIGURE 5B ACTIVATED SLUDGE
WITH SLUDGE STORAGE FOR F/M CONTROL

TABLE 3

COMPARISON OF ACTIVATED SLUDGE WITH AND WITHOUT INSTANTANEOUS F/M CONTROL

Parameter	No Control	Control
SRT days (Total Sludge Mass)	2.3	3.0
$(F/M)_I$	0.98 (Avg.)	0.71
Inflow gpm	20	20
Recycle gpm	10	variable
Effluent TOC mg/l	10.0	15.7
Effluent Turbidity JTU	26.6	53.1

F/M and one with control. As only one apparatus was available a truly controlled study could not be run, rather sequencial runs separated by several weeks were conducted. Thus parameters such as SRT and F/M could not be kept the same in both runs. With respect to maintenance of solids inventory and control of instantaneous F/M the model developed by Clemson University was verified. However, performance of the system was poorer when instantaneous F/M control was used.

The results indicate that the effluent TOC correlated with the effluent turbidity. Thus solids escaping the secondary clarifier were the prime cause of pollutants in the effluent. As the primary effluent inflow was kept constant in these runs effluent turbidity was probably due to hydraulic transient conditions in the secondary clarifier set up by variations in the sludge return rate needed to keep the instantaneous F/M constant.

Another study established to test the efficacy of instantaneous F/M control was conducted at the Hillsboro, Oregon sewage treatment plant. The Hillsboro treatment plant normally receives a typical domestic sewage but periodically receives shock loads of food processing wastes which produce upsets. Use of instantaneous F/M alone was not effective. However, a three pronged strategy using instantaneous F/M, aerator respiration rate (AERR) and short term sludge sedimentation testing was developed.

The Hillsboro plant is unique in that it is possible to recycle sludge or feed primary effluent to any combination of 4 compartments of the aerator. Thus the plant can be operated as a conventional activated sludge system with a variable aerator volume, a step aeration system or a contact stabilization system with variable proportions of stabilization and contact. The prime parameter real time control is the AERR. The AERR target is to be kept in the range of 12 to 15 mg/l per gram of MLSS. If the AERR increases the recycle and feed points are adjusted to operate deeper into the contact stabilization mode. If the AERR decreases below 12 the system is adjusted toward conventional activated sludge. The instantaneous F/M is kept between 0.15 and 0.35 with lower F/M values used as AERR increases and vice-versa. Sludge is wasted to maintain the SRT in a proper range and is modified depending on the results of the sedimentation test. Specific details of the control strategy are given else where (7). Table 4 presents a comparison of results prior to and subsequent to initiation of this control strategy. Figure 6 indicates the effect of maintaining the AERR in the proper range.

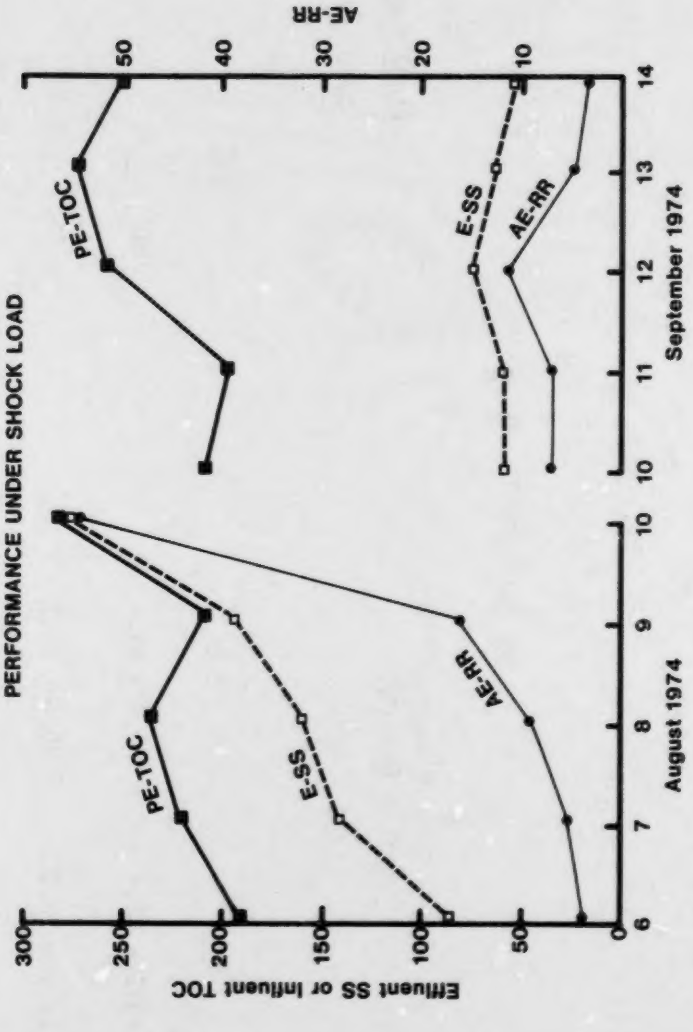
The results discussed above indicate that instantaneous F/M control is probably of limited utility for real time control of activated sludge systems. These data also indicate the utility of having some method of storing activated sludge other than in the secondary clarifier and of providing variable sludge conditioning time. The use of AERR to set the conditioning time appears valid; instruments to measure AERR are on the market. Finally these data indicate the need to pay more attention to the performance and design of the secondary clarifier. Indeed at Hillsboro even though the organic loading is lower during the rainy season, plant performance is worse than in the summer, (the shock load period) because of the higher hydraulic load on the clarifiers.

TABLE 4

PLANT PERFORMANCE (HILLSBORO)

	Prior to Control 1/72 - 6/73		Final Control 7/73 - 6/76	
	BOD	S.S.	BOD	S.S.
P.E.	149.7 ± 52.4	93.4 ± 14.3	149 ± 1	169.2 ± 117.8
F.E.	54.7 ± 20.0	70.0 ± 39.7	18.4 ± 8.3	25.1 ± 15.6
%R	58.9	17	83.9	78.4

FIGURE 6
EFFECT OF CONTROL OF AE-RR ON
PERFORMANCE UNDER SHOCK LOAD



b) Solids Handling

In an earlier section of this paper the performance of solids analysers used in conjunction with automated operation of a sludge thickener was discussed. The basic purpose of the study was to evaluate automated operation of the thickener incomparison to manual operation. Fortunately two full scale thickeners receiving indentical feeds were available for this study so that a control could be run. A total of 5 comparison runs were conducted each for a period of approximately 2 weeks. In each run the target level for the sludge blanket was different, the overall range covered was 3 feet below the surface to 8 feet below the surface. The results achieved were that automated control maintained the blanket level much closer to the target values than did manual control. However; this did not have any effect on the solids content of the thickened sludge, nor on its dewaterability (as measured by CST & Filter Leaf Tests). The advantage which did occur was that the solids in the overflow was significantly lower from the automated system. These data are presented in terms of solids capture efficiency in Table 5 . The MWCC has calculated that this reduction in solids return to the head end of the plant will save them $2400 per year per thickener. In addition labor savings per basin are estimated at $500 per year if automation is utilized. The estimated pay back period for the capital expenditure and installation labor to automate these thickeners is 9 months (8).

The project described above is only one of several on automation of sludge handling processes being conducted under an EPA grant to the MWCC; unfortunately none of the other projects has reached the point where significant data are being generated. One of the projects now nearing the intensive data gathering phase is a study of the automation of a vacuum filter. A full scale vacuum filter has been instrumented with the sensors listed in Table 6.

The vacuum filter control system consists of these monitoring devices, and the data collection and processing system with which they communicate. The monitors transmit data to a remote terminal (a microprocessor) which communicates with the central computer system. Control corrections determined by the central computer are output through the remote terminal to the process control devices. The central computer system is built around a sixteen bit, third generation mini-computer. The primary operator interface is the CRT and keyboard. A printer is available for generating written reports. A disc is used for long term storage of data for later analysis and for storage of system programs.

The software operating system is disc based and supports a high level programming similar to FORTRAN. The software consists of two main programs "application" and "control algorithm." The control algorithms software consists of the basic logic for controlling the sludge conditioning and vacuum filter processes and is readily changed at operator's console. The application software provides logic for operator interface and report generation, and allows easy manipulation of the process variables into several control loops for check out of alternative strategies in order to

TABLE 5

SOLIDS CAPTURE IN AUTOMATED AND MANUALLY OPERATED THICKENERS

Blanket Level Ft.	Solids Capture % Automated	Solids Capture % Manual
6.0	72	66
8.0	81	57
5.5	83	84
3	77	82
5	88	81

TABLE 6

SENSORS FOR VACUUM FILTER AUTOMATION

SENSOR	LOCATION OR USE
Magnetic Flowmeter	Sludge Flow to V.F.
Solids Monitor	Sludge Feed Concentration
pH	Sludge Prior to a After Conditioning
Thickness	Cake Thickness on Drum
Tachometers	Drum Speed
Turbidity	Filtrate Quality
Weight	Sludge Discharge Rate
Level	Filter Submergence
Cake Moisture	Degree of Dewatering

determine the most effective control strategy to implement in a total plant operating system.

The control strategies proposed for evaluation are summarized in Table 7. The first step in the evaluation is to determine the ability of the control system to maintain any variable at a selected value by manipulation of the control devices which will impact it. The effect of the variable on overall operation and cost of the process can then be determined by a series of tests which hold all stages of the process constant except that being evaluated.

The procedure for evaluating the control of the seven devices that affect vacuum filter automation consists of: a) an analysis of the device and the affected process variables, b) a series of open loop tests to determine the response of a change in a variable as a result of an input change, and c) a closed loop evaluation to determine the controllability of the variable by the controlling device. During the analysis step, the goal of control of the device is defined and the variables that it will affect are determined. Methods are choosen to hold constant all other functions of the filter process. The open loop tests are then defined and the device is subjected to a step change. The variables that will be affected are monitored on a recorder and trended by the computer. The recorder charts and trend reports are used to determine the effect that the change had on the process. The results of the open loop tests are used to empirically calculate the initial tuning constants for the closed loop control systems. Finally, the device is put on automatic control and the operation of the system is monitored. The controller tuning constants are adjusted to provide the best possible control. The response of the control system in respect to the initial objectives is then evaluated to determine the maximum variations that can be expected when operating under automatic control.

A diagram of the instrumented vacuum filter and its appurtances is given in Figure 7a & 7b. It is anticipated that the control strategy derived from these data will not require the extensive instrumentation as presented here. The data generated by this instrumentation will be used to formulate a viable model of the process from which the most cost effective control strategy will be determined. Once control of the vacuum filter is achieved work will begin on control of a multiple hearth sludge incinerator fed by this vacuum filter. It is anticipated that control of the vacuum filter will permit a sludge with a constant composition to be fed to the incinerator. This should result in lower fuel consumption, and less maintenance on the refractory.

Another control strategy project, which is just at the point of data generation, deals with the anaerobic digestion process. Andrews and his co-workers (9) have elucidated a proposed model for the performance of an anaerobic digestion system under non-steady state conditions. Figure 8 illustrates the mathematical model. Basically this model relates the rate of volatile acid fermentation, and methane fermentation to a variety of kinetic constants and physical and chemical parameters. In this project an attempt will be made to verify this model both qualitatively and then quantitatively. The latter will involve determination of the kinetic

TABLE 7

SUMMARY OF PROPOSED CONTROL LOOPS FOR VACUUM FILTER AUTOMATION

CONTROL LOOP	FEEDFORWARD	FEEDBACK
Sludge Feed Rate	-	Filter Vat Level
Drum Speed	Solids Feed Rate	Cake Thickness
Drum Speed	Solids Feed Rate	Cake Moisture
Vat Agitator Speed	Solids Feed Rate	Cake Thickness, Cake Mositure, Filtrate, Turbidity
Ferric Chloride Feed	Solids Feed Rate	Cake Moisture
Ferric Chloride Feed	Solids Feed Rate	pH of ($FeCl_3$ + Sludge)
Ferric Chloride Feed	Solids Feed Rate	Filtrate Turbidity
Lime Feed	Ferric Flow	pH of Conditioned Sludge
Conditioning Tank Mixer	Velocity Gradient Calculation	Filtrate Turbidity
Form Vacuum	Drum Speed	Form Vacuum, Cake Thickener
Form Vacuum	Solids Feed Rate	Form Vacuum, Cake Thickness, Cake Mositure

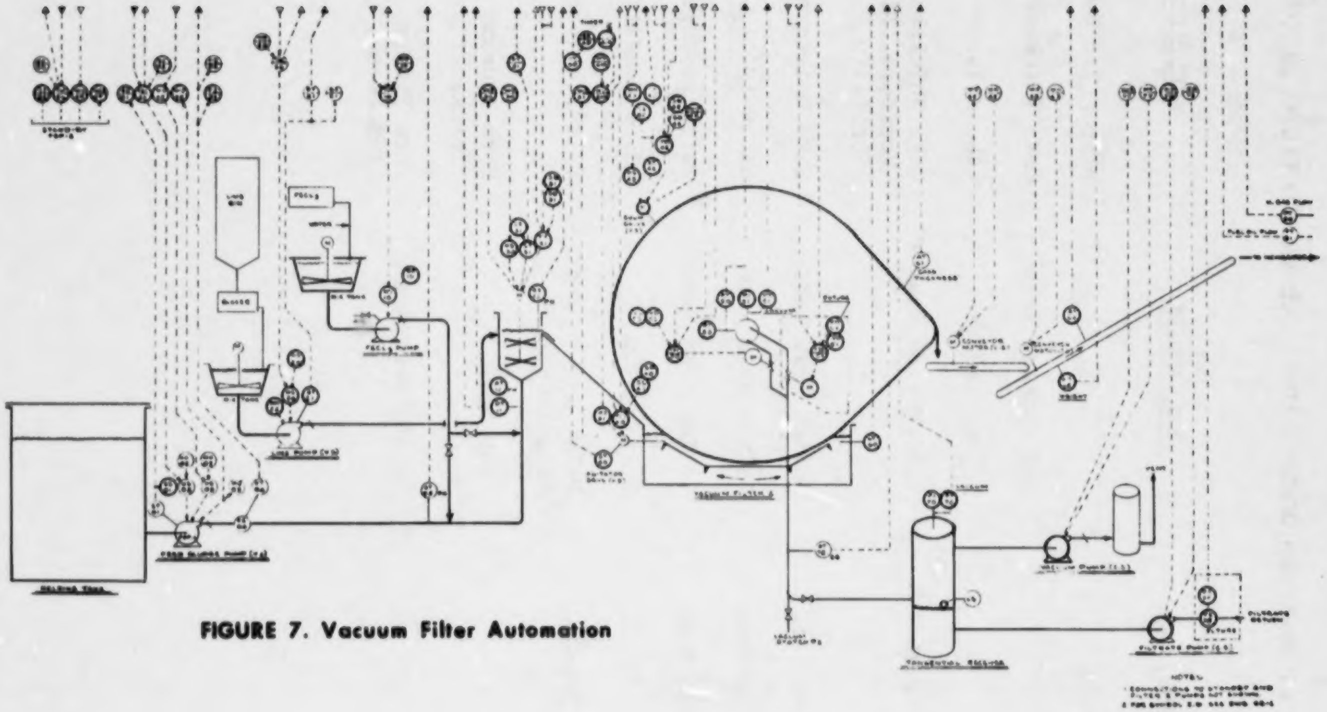


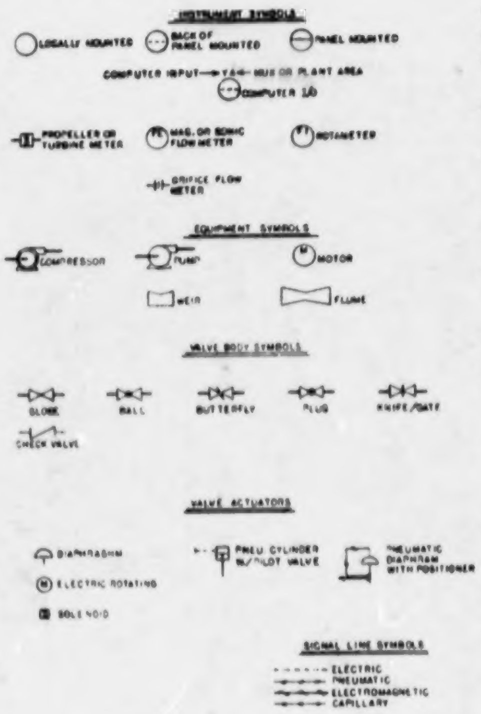
FIGURE 7. Vacuum Filter Automation

INSTRUMENT LETTER IDENTIFICATION

AE	ANALYSIS PRIMARY ELEMENT
AI	ANALYSIS INDICATE
AIT	ANALYSIS INDICATE TRANSMIT
ASH	ANALYSIS SWITCH HIGH
ASHH	ANALYSIS SWITCH HIGH HIGH
FE	FLOW RATE PRIMARY ELEMENT
FI	FLOW RATE INDICATE
FQ	FLOW RATE TOTALIZER
[illegible]	FLOW RATE TRANSMIT
FY	FLOW RATE RELAY
GT	POWER FACTOR TRANSMIT
GI	POWER FACTOR INDICATE
HC	HAND CONTROL STATION
HIC	HAND INDICATE CONTROL STATION
HS	HAND (MANUAL) SWITCH
IP	CURRENT/PRESSURE
IT	CURRENT TRANSMIT
LI	LEVEL INDICATE
LSH	LEVEL SWITCH HIGH
LSHH	LEVEL SWITCH HIGH HIGH
LSL	LEVEL SWITCH LOW
LSX	LEVEL SWITCH EXCESS
LT	LEVEL TRANSMIT
MIL	MOTOR INDICATE LIGHT
MSH	MOTOR SWITCH START/STOP

QAH	TORQUE ALARM HIGH
QSH	TORQUE SWITCH HIGH
QSX	TORQUE SWITCH EXCESS
PI	PRESSURE INDICATE
PIC	PRESSURE INDICATE CONTROL
PS	PRESSURE SWITCH
PY	PRESSURE RELAY
PT	PRESSURE TRANSMIT
SC	SPEED CONTROL
SI	SPEED INDICATE
ST	SPEED TRANSMIT
TSH	TEMPERATURE SWITCH HIGH
TY	TEMPERATURE RELAY
VSC	VALVE SWITCH CLOSE
VSO	VALVE SWITCH OPEN
VSFC	VALVE SWITCH FAILURE CLOSE
VSFO	VALVE SWITCH FAILURE OPEN
XSH	VIBRATION SWITCH HIGH
XY	VIBRATION RELAY
YSC	COMPUTER SWITCH CONTROL
ZI	POSITION INDICATE
ZSC	POSITION SWITCH CLOSED
ZSO	POSTION SWITCH OPEN
ZT	POSITION TRANSMIT

P&ID LEGEND OF SYMBOLS



COMPUTER I/O LETTER IDENTIFICATION

AH*	ANALYSIS HIGH
AT	ANALYSIS TRANSMIT
AX	ANALYSIS EXCESS
FT	FLOW TRANSMIT
GT	POWER FACTOR TRANSMIT
HC	HAND (MANUAL) CONTROL
HS	HAND SWITCH
IT	CURRENT TRANSMIT
TT	TEMPERATURE TRANSMIT
MM	MOTOR OPERATE
MN	MOTOR START/STOP
MF	MOTOR FAILURE
QH	TORQUE HIGH
QX	TORQUE EXCESS
PH	PRESSURE HI
PL	PRESSURE LOW
PT	PRESSURE TRANSMIT
SA	SPEED ALARM
SB	SPEED DECREASE
SC	SPEED CONTROL
SD	SPEED INCREASE
ST	SPEED TRANSMIT
TX	TEMPERATURE EXCESS
VB	VALVE CLOSE
VC	VALVE CONTROL
VO	VALVE OPEN
VF	VALVE FAILURE

Y	COMPUTER CONTROL
YZ	POSITION DRIVE
ZC	POSITION CLOSED
ZO	POSITION OPEN
ZI	POSITION INDICATE
ZT	POSITION TRANSMIT
XX	VIBRATION EXCESS

FIGURE 7a. Legend For Figure 7.

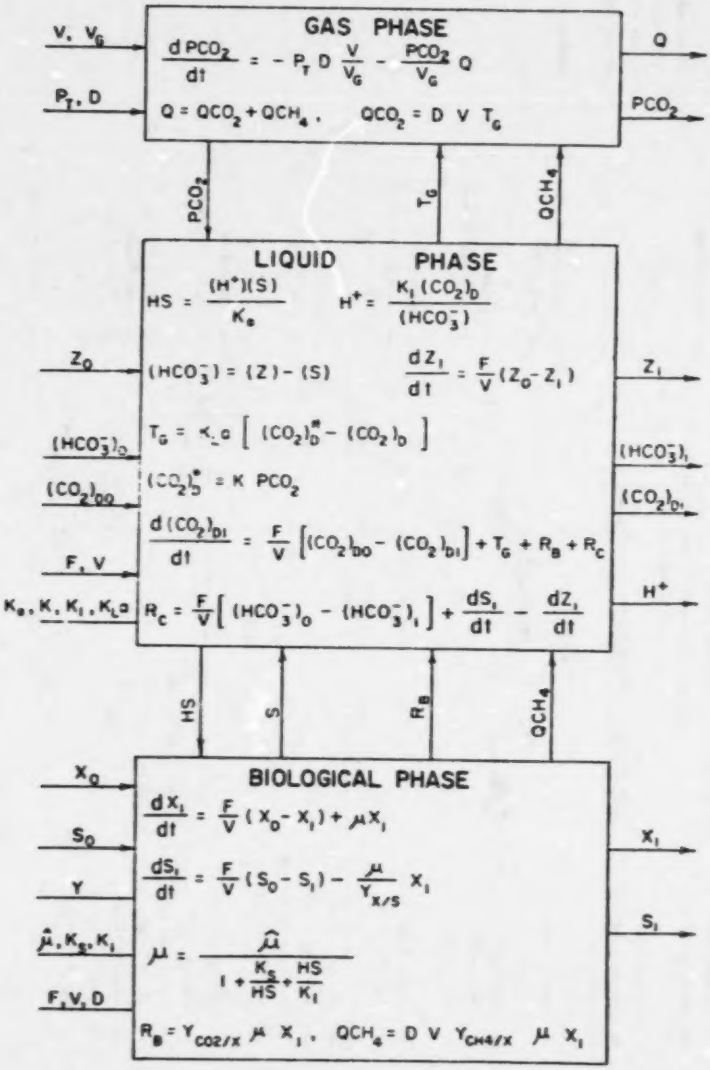

GAS PHASE
$\frac{dPCO_2}{dt} = -P_T D \frac{V}{V_G} - \frac{PCO_2}{V_G} Q$
$Q = QCO_2 + QCH_4, \quad QCO_2 = D\ V\ T_G$
V, V_G
P_T, D
Q
PCO_2
PCO_2
T_G
QCH_4
LIQUID PHASE
$HS = \frac{(H^+)(S)}{K_a}$
$H^+ = \frac{K_1 (CO_2)_D}{(HCO_3^-)}$
$(HCO_3^-) = (Z) - (S)$
$\frac{dZ_1}{dt} = \frac{F}{V}(Z_0 - Z_1)$
$T_G = K_La \left[(CO_2)_D^* - (CO_2)_D \right]$
$(CO_2)_D^* = K\ PCO_2$
$\frac{d(CO_2)_{D1}}{dt} = \frac{F}{V}\left[(CO_2)_{D0} - (CO_2)_{D1}\right] + T_G + R_B + R_C$
$R_C = \frac{F}{V}\left[(HCO_3^-)_0 - (HCO_3^-)_1 \right] + \frac{dS_1}{dt} - \frac{dZ_1}{dt}$
Z_0
$(HCO_3^-)_0$
$(CO_2)_{D0}$
F, V
K_a, K, K_1, K_La
Z_1
$(HCO_3^-)_1$
$(CO_2)_{D1}$
H^+
HS
S
R_B
QCH_4
BIOLOGICAL PHASE
$\frac{dX_1}{dt} = \frac{F}{V}(X_0 - X_1) + \mu X_1$
$\frac{dS_1}{dt} = \frac{F}{V}(S_0 - S_1) - \frac{\mu}{Y_{X/S}} X_1$
$\mu = \frac{\hat{\mu}}{1 + \frac{K_S}{HS} + \frac{HS}{K_I}}$
$R_B = Y_{CO2/X}\ \mu\ X_1, \quad QCH_4 = D\ V\ Y_{CH4/X}\ \mu\ X_1$
X_0
S_0
Y
$\hat{\mu}, K_S, K_I$
F, V, D
X_1
S_1

FIGURE 8

MATHEMATICAL MODEL FOR ANAEROBIC DIGESTION SYSTEM constants for the production of volatile acids under various loading conditions. Once this model is verified or modified it can be utilized to predict the situations of loading which will produce failure in an anaerobic digestion system. This prediction can then be used to elucidate control strategies to prevent or cope with digester failure.

The experimental equipment to be used includes two completely mixed, temperature controlled anaerobic digestion units. One unit of 245 liter capacity will be operated at steady state conditions to act as the source of digesting sludge for the experiments. The other unit is a highly instrumented digester in which experiments will be conducted. Figure 9 is a schematic of the experimental unit illustrating the parameters which will be monitored. The only parameters which will not be automatically monitored will be alkalinity and volatile acids; however, sampling of sludge so that manual analyses can be conducted is automated.

Sludge from the seed unit will be transferred to the experimental unit and then stressed in a variety of ways. The system may receive a loading shock, a pH shock, a temperature shock or a toxic agent. The unit will be monitored for the various model parameters and the data will be automatically entered into an HP-98-25 desk top computer. Figure 10. The model has been loaded into this computer so that data analyses can be conducted in conjunction with data generation.

Control strategies developed in this study will be tested in a full scale anaerobic digester at the Anoka Treatment Plant of MWCC.

III. Computer Applications

A general area of much work in the future will involve the application of digital computers to the management of the treatment plants, and area wide pollution control systems.

a) Individual Treatment Plant Control

A state of the art monitoring, control and management information system using a digital computer has been installed and is under evaluation at the Central Contra Costa Water Reclamation Plant near Concord, California. This plant has been extensively described elsewhere (19). It is a 30 MGD AWT plant featuring lime precipitation of raw sewage, primary sedimentation, single stage biological oxidation of carbon and ammonia, secondary sedimentation, and effluent filtration. The effluent will be used for industrial cooling water. Solids handling includes: 2 stage centrifugation, recalcination of the first stage cake with dry lime classification, and incineration of the second stage cake. Solids waste plus sludge pyrolysis may be substituted in the future for the second stage cake incineration. A description of the instrumentation, and automation is given below.

As shown in Figure 11, the plant monitoring, control, and management information system comprises various process sensors and final control elements and a distributed, three-level network of digital computer systems. Hardware and software elements are described separately hereunder.

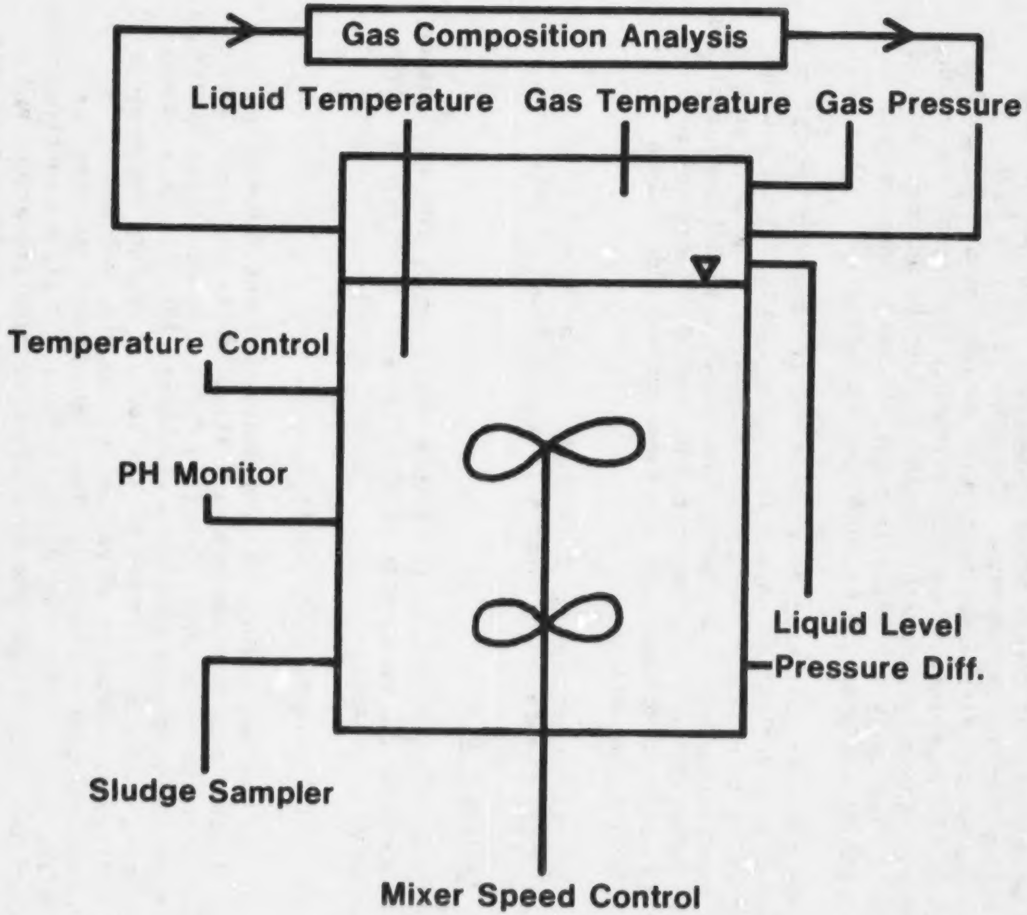


FIGURE 9 ANAEROBIC DIGESTION MONITORING AND CONTROL

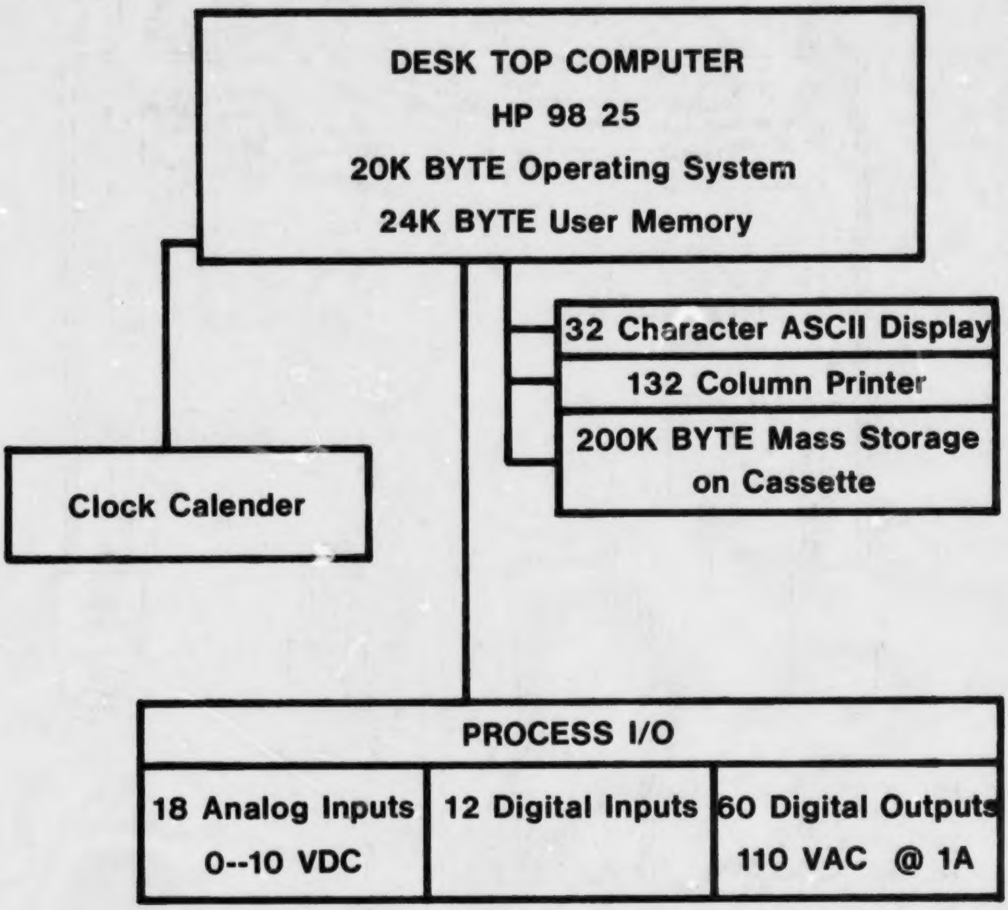
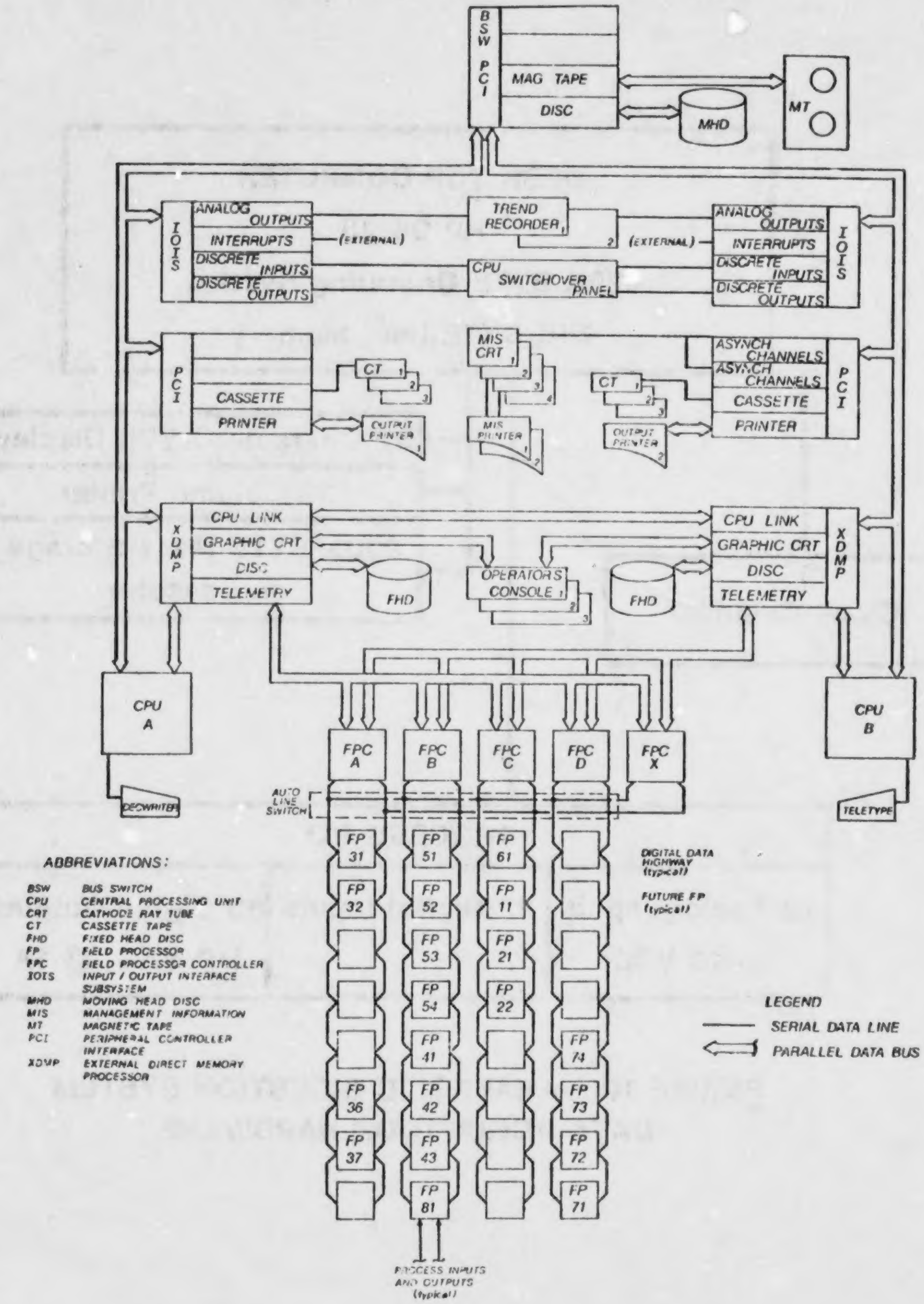


FIGURE 10 ANAEROBIC DIGESTION SYSTEM
DATA ACQUISITION HARDWARE

FIGURE 11

PLANT MONITORING CONTROL AND M.I.S. DIAGRAM



ABBREVIATIONS:

BSW	BUS SWITCH
CPU	CENTRAL PROCESSING UNIT
CRT	CATHODE RAY TUBE
CT	CASSETTE TAPE
FHD	FIXED HEAD DISC
FP	FIELD PROCESSOR
FPC	FIELD PROCESSOR CONTROLLER
IOIS	INPUT / OUTPUT INTERFACE SUBSYSTEM
MHD	MOVING HEAD DISC
MIS	MANAGEMENT INFORMATION
MT	MAGNETIC TAPE
PCI	PERIPHERAL CONTROLLER INTERFACE
XDMP	EXTERNAL DIRECT MEMORY PROCESSOR

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Hardware

Conventional physical and analytical process sensors are employed to monitor pertinent process variables, and all outputs are converted to 4-20 mA. Modulating final control elements accept either 3-15 psig or 4-20 mA signals for position or speed modulation. A solid state digital-to-analog converter is provided for each analog output channel. Incremental raise/lower signals are fed to each converter from the associated field processor described below, and the converter in turn increments or decrements the 4-20 mA analog output to the associated final control element. All discrete status sensors are monitored, and three-wire control is provided for all final control elements requiring on/off or open/close signals such as pumps and valves. Local manual controls are provided for maintenance purposes.

Twenty field processors (FPs) are located at strategic points throughout the plant, and five field processor controllers (FPCs) are located in the plant control center. Hardwired connections are provided between the FPs and all process sensors and final control elements, and a digital data highway connects each group of FPs to the associated FPC.

The data acquisition and control transmission system is designed to relieve the on-line central processing unit (CPU) of data acquisition and preprocessing duties and final element control duties. A brief discussion of the functions performed at each level of the hierarchy follows:

Level 1 - The FPs perform data acquisition, preprocessing, transmission, and command execution functions upon request from the associated FPC. The FPs perform the following specific functions: (a) monitor process sensors and control process regulators; (b) digitize and standardize all analog inputs; (c) perform limit checking on all analog inputs, including comparison of analog input values with instrument reasonableness limits and standardized analog values with program adjustable upper and lower amplitude, rate of change, and delta band limits; and (d) upon request from the associated FPC, perform exception reporting by transmitting analog and discrete sensor inputs that have changed status since the previous scan.

Level 2 - The FPCs serve as the interface between the FPs and the on-line CPU. The FPCs are accessed by the on-line CPU through a data highway controller which in turn interfaces directly to CPU memory via an external direct memory processor. The FPCs manage the scanning of process sensors in accordance with the schedule established by the on-line CPU, report data input in a blocked, edited format to the on-line CPU upon demand, and direct commands to the FPs addressed by the on-line CPU. The FCPs also maintain communications, security and integrity with the FPs described above.

Level 3 - The on-line CPU controls the general timing of data acquisition by modifying instrument scan rates in response to operator requests and services "data ready" interrupts from the FPCs to transfer blocked, edited data into memory. All final processing of data, such as activation of alarm annunciation and transfer to the process data storage area in memory, together with control over all final control elements, is maintained at this level.

All plant control center equipment, with the exception of one operator console module, is located in the Plant Operations Building.

The operator's console in the plant control center consists of two color graphic CRT display modules, one alphanumeric CRT display module, and two printer-plotter units. The console is designed to permit the operator not only to rapidly access and display the desired information in pictorial, graphical, or alphanumeric format, but also to respond to and correct abnormal process operating conditions in a timely manner. Provision is made for the operator to override any control strategy and manually control any final control element.

The overall console design concept was to provide a single interface that permits all operator functions to be within reach of a seated operator and to be accomplished in a timely and orderly fashion. The goal was to provide an interface that raised the performance level of the average operator through enriched information availability and improved presentation as compared to a conventional control panel. The following design features were specified and subsequently provided:

1. Three CRT monitors and keyboards to permit both parallel and serial information transfer to the operator in graphical, pictorial, and for textual format.

2. A management by-exception operating principle to provide the operator with faster communication of incipient problems and results of restorative or corrective action.

3. A display hierarchy that permits the operator to rapidly access the level (i.e., from a plant overview display to details about a single parameter) and format of the information he desires.

4. The maximum use of lightpens and function pushbuttons to facilitate operator-display interaction.

5. A display system that employs a flexible architecture so that changes and additions can be made on-line without any changes in hardware.

Software

Both computer systems are provided with a real-time operating system (RTOS) because either system can be assigned to real-time process control. The RTOS operating on the on-line system (i.e., the computer system performing real-time process control) is configured to support (a) master control of the system, (b) regulatory and sequence control of the process, (c) data communications over the digital data highway (d) data processing of all process-related data (e) automatic logging and annuniciation of system alarms and events, (f) man-machine interaction via the CRT's and printers, and (g) historical data storage. The RTOS operating on the off-line system is configured to support (a) local job processing, such as assemblies and compilations; (b) remote job entry to the on-line CPU; (c) the management information system; (d) periodic data transmission from the on-line computer system in preparation for mode switchover in the event of a major system component failure; (e) an extensive mathematical and statistical subroutine library; and (f) historical information retrieval. Simplified block diagrams of the on-line and off-line system software are presented in Figures 12 and 13, respectively. The major application programs are described below.

Control Strategy Program

The key software subsystem for real-time control of the plant processes is the control strategy program which employs the "building block" approach. The control strategy program employs a standard set of control algorithms that emulate hardware control and logic functions. The majority of the control strategies for both regulatory and sequence control are implemented with the above user-oriented program. The balance of the control strategies are special purpose and are written in Fortran IV. The control strategies written in Fortran are callable as subroutines by the control strategy program.

Basically, the control strategy program comprises separate preprogrammed subsystems for data acquisition, data management, control actions, console displays, and process output control. Within these subsystems, the user provides data for a given control strategy via a CRT-displayed questionnaire. A pre-defined system uses these data to link up the data acquisition section with the process, configure the data base properly, specify control actions and timing, define the format of operator outputs, and link up the actual outputs to the process.

A given control strategy can be designed by specifying the proper combination of control algorithms via an alphanumeric CRT connected to the off-line system. The user provides the parameters for each algorithm by responding to a CRT-displayed questionnaire.

The control strategy program automatically constructs the process control data base from the data provided in the questionnaire. If the user enters unreasonable data or fails to answer all questions, the program notifies the user via CRT-displayed messages. Following acceptance of the input data, the control strategy program performs the compilation of the data into binary form. The data are then transferred to the data base that resides within the on-line system for implementation. In actual operation, all control strategy calculations are performed by a master program which scans the data base loop records sequentially and performs the operations required at that time by each algorithm.

Console Display Generation Program

Because the operator console CRT displays are under program control display formats can be reconfigured, and new formats can be added without

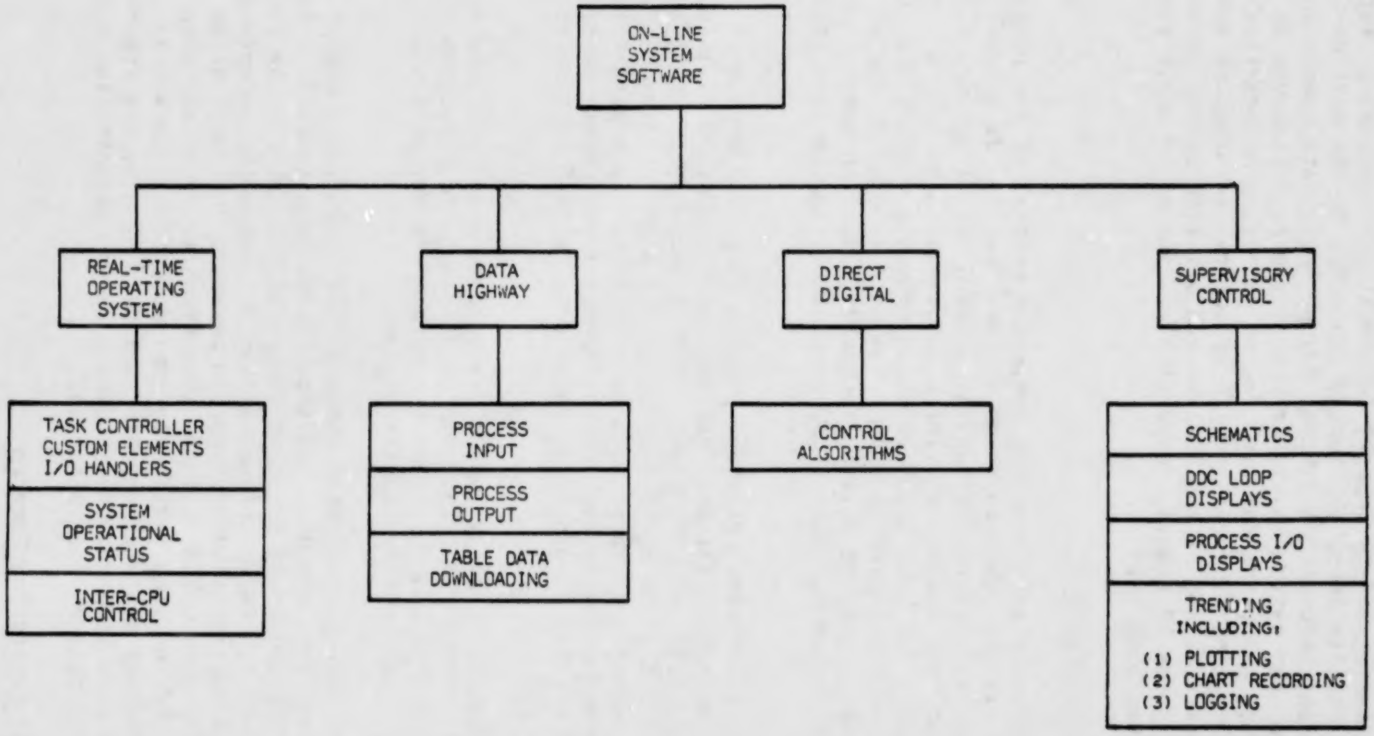

FIGURE 12 ON--LINE SYSTEM SOFTWARE

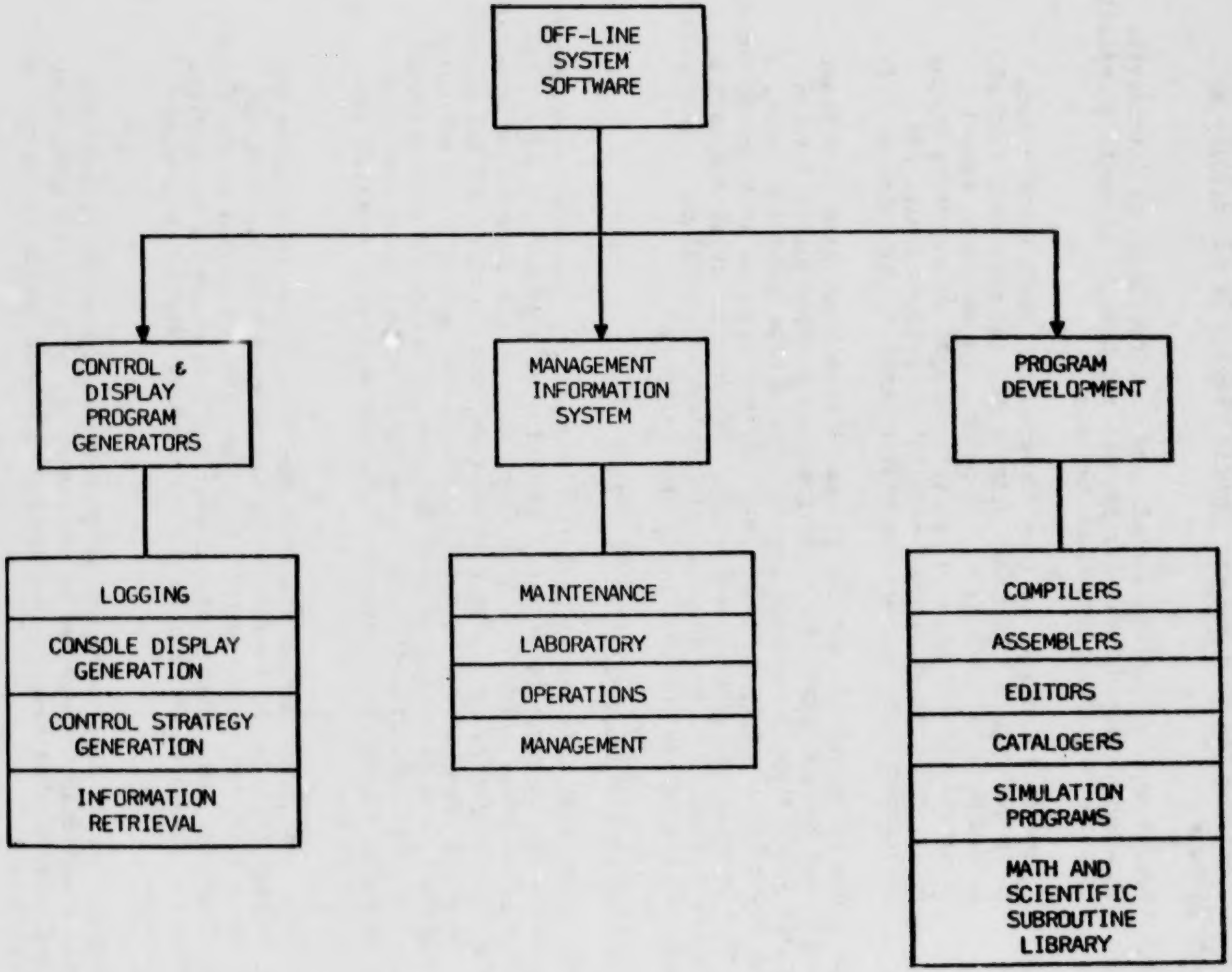

FIGURE 13 OFF--LINE SYSTEM SOFTWARE

any modifications or addition of hardware. An interactive, display generation program is provided for this purpose and permits the on-line creation, modification, display and dynamic updating of color graphic CRT displays. Construction of a dynamic graphic display is essentially a two-step process:

1. From a menu of process symbols and line sections, an interactive background display compiler permits the user to generate a typical graphic display in less than one hour.

2. A semaphore compiler is then employed to permit dynamic data to be added to a background display for any measured, virtual, or manipulated variable or set-point. In addition, symbol color changes are used to indicate changes in operating status (e.g., red--running, green--available, yellow--fault, etc.) and current control mode (automatic, manual, provision or off).

As shown in Figure 14, the CRT display system associated with plant monitoring and control functions is arranged in a hierarchical fashion; the higher level displays provide general information about the plant, and the lower level displays provide increasingly detailed information about plant unit process and control subsystems. The highest level displays are accessed by pushbuttons, and lower level displays are obtained by successive lightpen strikes on the appropriate display targets.

Historical Data Storage and Retrieval System

The historical data storage and retrieval system provides the operator with a means for storing, retrieving, reviewing, and processing data. The historical data base includes data relating to past plant operating conditions in terms of alarm/event data, operator entries, and a compressed log of past plant operating data, including analog, virtual and manual input variables. The historical data base is also used to store discarded control strategy programs, old CRT schematic displays, previously issued plant operating report formats, historical maintenance data and any other archival data saved by the user.

A delta band data compression technique is employed to minimize the memory storage required to record analog and virtual variable data and manual entry date. The most recent historical data is stored on the fixed-head disc memory; the contents are automatically transferred to cassette tape when the allocated storage on the disc memory reaches 80 percent utilization.

The historical data retrieval program is configured to receive the retrieval requirements specified by the operator, process the historical files and transfer the results to a specified output device in textual or graphical format.

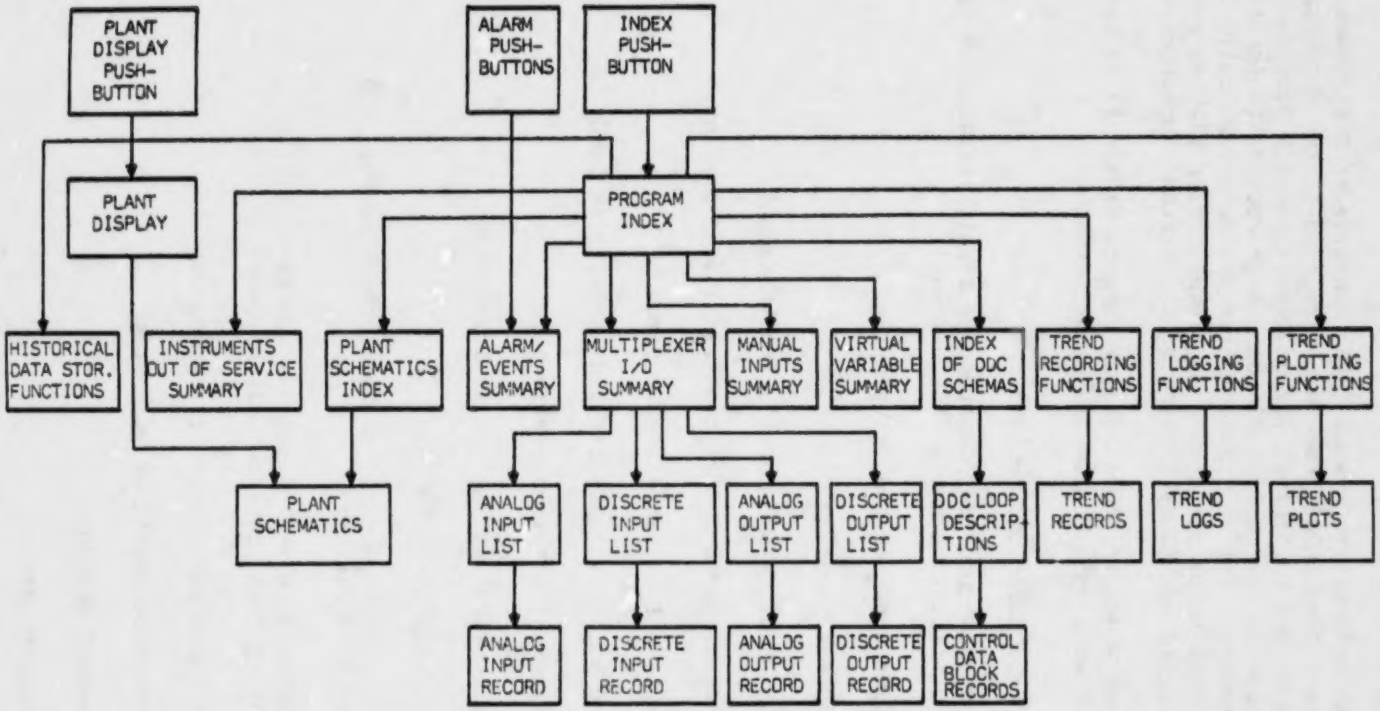

FIGURE 14 PLANT MONITORING AND CONTROL HIERARCHY

OPERATORS CONSOLE

Management Information System

The management information system (MIS) consists of a software "package" of Fortran programs called SCORE that were furnished by the Municipal Equipment Division of the Envirotech Corporation. The MIS program operates on the off-line computer system; the MIS data base resides in the 12 megaword moving head disc memory. The on-line computer system periodically updates the MIS data base with information such as process drive elapsed running times and daily integrated values for analog and virtual variables.

The MIS program shown in block diagram form in Figure 15 is designed to provide the following information for plant personnel:

(a) Maintenance Repair Information

A series of programs provides maintenance repair information in response to an equipment identification number as follows:

1. Equipment number and description
2. Equipment location
3. Location of manufacturer's repair manual
4. Component parts description (including part no.)
5. Inventory location of spare parts
6. Spare parts availability and quantity on hand
7. Location of interchangeable parts
8. On-site capability of parts fabriaction
9. Lubrication requirements
10. Parts Vendors - name, address, phone number, and parts nomenclature
11. Outstanding parts on order - supplier, part no., quantity, and promised delivery date

The following forms and reports are produced:

1. Maintenance permit or work order
2. Inventory drawdown
3. Inventory update
4. Equipment data update

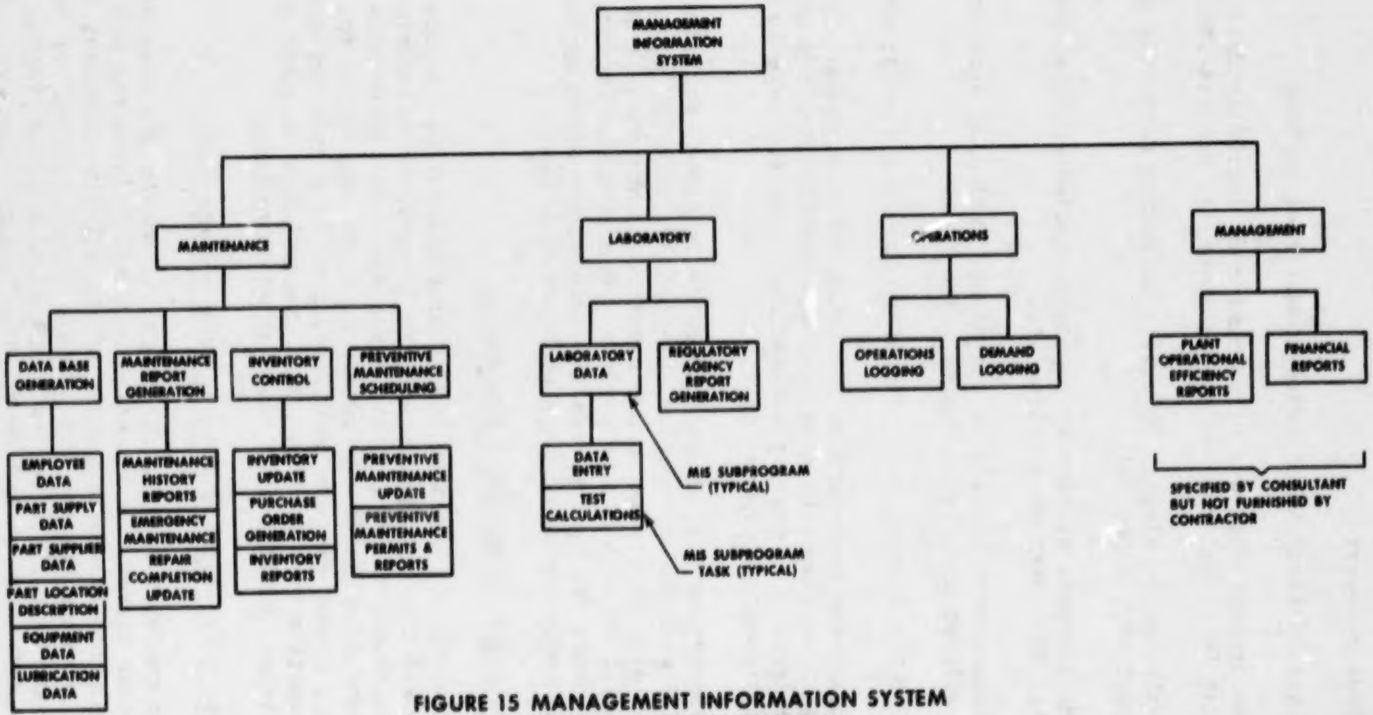
MANAGEMENT INFORMATION SYSTEM
MAINTENANCE
LABORATORY
OPERATIONS
MANAGEMENT
DATA BASE GENERATION
MAINTENANCE REPORT GENERATION
INVENTORY CONTROL
PREVENTIVE MAINTENANCE SCHEDULING
EMPLOYEE DATA
PART SUPPLY DATA
PART SUPPLIER DATA
PART LOCATION DESCRIPTION
EQUIPMENT DATA
LUBRICATION DATA
MAINTENANCE HISTORY REPORTS
EMERGENCY MAINTENANCE
REPAIR COMPLETION UPDATE
INVENTORY UPDATE
PURCHASE ORDER GENERATION
INVENTORY REPORTS
PREVENTIVE MAINTENANCE UPDATE
PREVENTIVE MAINTENANCE PERMITS & REPORTS
LABORATORY DATA
REGULATORY AGENCY REPORT GENERATION
DATA ENTRY
TEST CALCULATIONS
MIS SUBPROGRAM (TYPICAL)
MIS SUBPROGRAM TASK (TYPICAL)
OPERATIONS LOGGING
DEMAND LOGGING
PLANT OPERATIONAL EFFICIENCY REPORTS
FINANCIAL REPORTS
SPECIFIED BY CONSULTANT BUT NOT FURNISHED BY CONTRACTOR

FIGURE 15 MANAGEMENT INFORMATION SYSTEM PROGRAM ORGANIZATION

5. Repair report

6. List of parts inventories, quantities on hand

7. Maintenance history by account code including date, tag No., employee No. labor, downtime, process status, etc.

8. Maintenance history by skill, including date, tag No., part No., labor, etc.

9. Maintenance history by employee, including date, report No., tag No., part No., hours, etc.

10. Maintenance history by tag number including date, report No., employee No., labor, parts, etc.

11. Trouble listing on equipment by tag No., account code, etc.

12. Equipment description, including where equipment is used, what equipment is used for, who manufactured equpment, general equipment specifications, cost code applied to equipment, equipment name

13. Preventive maintenance currently required, including: all past due preventive maintenance, priority of preventive maintenance; date when preventive maintenance is due, trade skill requirments to perform preventive maintenance, unique numbers for each preventive maintenance activity due, a one sentence description of each activity due.

(b) Preventative Maintenance Scheduling

Preventive maintenance scheduling is provided for all process equipment. Elapsed running time is used to trigger the automatic scheduling of preventive maintenance. Preventive maintenance information is provided in response to inputs of a present or future date and one of the following: An equipment No., a unit process identification, or a code representing all outstanding preventive maintenance. Upon completion of a given preventive maintenance activity, current file information is updated.

(c) Process drive elapsed running time computation

The on-line computer system continuously computes the elapsed running time of all process drives. The on-line computer updates the MIS data base every ten minutes by communicating to the off-line computer system. Discrete inputs indicate whether the process drives are "on" or "off". Minute counters are maintained for each drive and a daily program updates corresponding "hour" counters and resets the "minute" counters. A daily "compare" program compares the accumulated hourly counts against contents of memory locations which contain maintenance "service interval" information.

Depending on manufacturer's recommendations, each equipment item has one or more maintenance tasks to perform.

(d) Inventory Control Reports

Inventory control reports of replacement parts and materials include the following:

1. Audit routines, including minimum reorder points; current quantities; costs; location of parts.
2. Update routines to reflect decreases and increases.
3. Purchase order/requisition routines to provide recommended reorder quantities. In addition, up to three possible suppliers are listed by name, address, and phone number, purchase order terms are printed, and the information is filed for subsequent cross reference by the maintenance report routines.

(e) Management Information Reports

Management information reports include:

1. Plant operations/efficiency reports including equipment down time, personnel, trend analysis, and budget levels.
2. Financial Reports - Financial reports are based on data from operational and maintenance routines. Routines summarizing labor and repair costs are provided and permit several formats classifying cost information as follows: specific equipment, specific employee, specific part, unit operations, and the entire facility. Labor costs are subdivided to include straight time pay rates, overtime pay rates, and holiday pay rates.

The evaluation will include how cost effective this state-of-the-art system is for application to wastewater treatment plants.

b) Areawide Management

The concept of areawide management of a number of treatment plants with the aid of a central digital computer linked to a small computer located at the satellite plants is being tested in Cuyahoga County, Ohio (11). Actually the system which will be tested will be much simpler in concept. It will employ the installation of field proven reliable sensors at one treatment plant which will be interfaced to a programmable logic controller (PLC) capable of monitoring the sensors and calling an operator upon a plant upset or mechanical breakdown.

The monitoring system sensors which will be installed at the Richmond Park Terrace Treatment Plant are detailed in Table 8. This plant is a 200,000 gpd extended aeration package plant with effluent filtration.

The PLC will be used to perform several control functions in addition to monitoring. These will include: cycling pumps and blowers, starting stand by pumps or blowers in case of a failure, and starting and stopping blowers in phase with changes in flow to the plant.

An analysis will be performed to determine if this technique is superior to one or more other management alternatives such as regionalization of plants, independent plant operation and regular circuit rider operation. If this concept proves out a study using a process control computer at a central site with a microprocessor placed at each plant will be initiated. The sensors used in this instance will be those required for process control.

c) "Desk Top" and "Smart Computers"

The newest area of research activity which is planned takes into account the newest generation of computer technology. These are the so called "desk top computers". These devices have sufficient capacity to operate small to medium size treatment plants but cost significantly less than present state-of-the-art computers. In addition they offer man machine interaction. An example is the anaerobic digestion control strategy project described above. The model fits easily on a programable calculator. A treatment plant operator could once the model is verified for his plant use this system to predict when trouble would arise as conditions change at the plant. Another example is the Hillsboro, Oregon plant. Programable calculators are used by the operators to determine when to shift the mode of operation. The operator inputs specific sensor data and the calculator prints out operational instructions.

Finally a variation called the "intelligent computer" is now coming on the market. These inexpensive devices do not have large or rapid computational power but can learn by trial an error. The concept to be investigated is to place one of these devices at a plant for a year and periodically feed in operational data and decisions of the operational staff. The computer would gradually generate an emperical control strategy for the plant based on trial and error. The man-machine interaction in this case would develop a "trusting" relationship during this period of time, thus overcoming resistance to automation.

TABLE 8

SENSORS INSTALLED AT RICHMOND PARK TERRACE

NO. SENSORS	TYPE OF SENSOR	INTERFACED TO PLC ALARM MONITOR
Pump Station		
1	Liquid Level	High & Low Level
2	Power Demand[1] of 2 Pumps	No Power
2	Bearing Temperature of 2 Pumps	High Temperature
Extended Aeration		
3	Discharge Pressure of 3 Blowers	Low Level
3	Power Demand[1] of 3 Blowers	No Power
3	Bearing Temperature of 3 Blowers	High Temperature
2	Sludge Blanket Detector	High Level
General Alarms		
1	Smoke or Fire	1
1	Grounds Security	1
1	Chlorine Gas Leak	1
Total Sensors = 19		Total Alarms = 20

[1] Power Demand Will Monitor Contact Closure On Magnetic Motor Starter.
All Sensors Alarms Will Be Form C Contact Closure.

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A SYSTEM FOR TANNERY EFFLUENT TREATMENT

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ABSTRACT

In order to satisfy stream discharge requirements at its Winchester, N. H. chrome tan shearling tannery, A. C. Lawrence Leather Company selected primary and secondary systems which were and are unique as applied to tannery effluent treatment in the United States. The primary clarification is accomplished by means of coagulation and flotation, using electroytic as well as mechanical microbubble generation. The secondary biological section is a so-called CARROUSEL, a technical modification of the Passveer oxidation ditch.

The U. S. Environmental Protection Agency provided financial support to the operation of the facility for a period of twelve months. During that time complete analytical data representing winter as well as summer operating conditions was acquired and operating cost data was accumulated. This report also describes the design and operation of that system, and makes note of the possibility of application of the same principles to other tannery wastewaters.

This project has been financed in part with federal funds from the Environmental Protection Agency under grant number S804504. The contents do not necessarily reflect the views and policies of the Environmental Protection Agency, nor does mention of trade names of commercial products constitute endorsement or recommendation for use.

A SYSTEM FOR TANNERY EFFLUENT TREATMENT

Some tanneries are fortunate enough to be located geographically or otherwise so that the inevitable impact of effluent pollution control is minimal. Others have to go the full distance and design, construct, and operate wastewater treatment facilities of their own, to meet State or Federally imposed discharge requirements. This we were required to do at one of our A. C. Lawrence tanneries. We have just completed a demonstration project for the U. S. Environmental Protection Agency on this system, in which the Agency supported the operation for a twelve month period, and during which a significant amount of data was accumulated, representative of both summer, and winter, operating conditions. I am using excerpts from the report on the E.P.A. project to describe that system to you today. Before going further I would like to acknowledge the aid and assistance of two consultants, Dr. E. R. Ramirez of Swift Environmental Systems, Chicago, and Dr. W. L. Zamitis, P. E. of Envirobic Systems, Inc., New York. This paper contains some excerpts from material written by them as pertaining to their particular fields, Dr. Ramirez in the primary treatment phase, and Dr. Zemaitis in the secondary.

Introduction

Tanneries in general are pollution intensive industrial complexes, generating comparatively large volumes of high concentration wastewater which historically have been discharged into rivers and waterways without much, if any, attempt toward purification. This paper is intended to present the details of construction, and operating data, including analytical results, for a system which was designed to eliminate most of the objectionable components of a tannery effluent formerly discharged directly into a small river.

Tanneries are not all alike. The basic design of procedures for hide preparation, tanning and finishing vary rather widely according to the kind of raw hides employed and the characteristics desired in the finished leather product. Accordingly the U. S. Environmental protection Agency, in order to better understand the problems associated with pollution abatement, classified the various segments of the industry into seven categories. These are:

Category 1. Cattlehide - Pulp Hair - Chrome Tan
2. Cattlehide - Save Hair - Chrome Tan
3. Cattlehide - Non Chrome Tan
4. Thru-the Blue
5. Retan only
6. No Beamhouse Tannery
7. Shearlings

In this presentation the category is number seven, Shearlings.
I hasten to say, at this point, that I realize full well that all or most of you are not in the business of running a shearling tannery. Please do not immediately arrive at the conclusion that this does not apply to you. The principles employed are applicable to any tannery effluent as I will point out later on.

A "shearling" tannery is one which tans and finishes sheep pelts with the wool intact, thus producing "skins" which, except for alterations of character and appearance to accomplish permanent preservation, and enhance aesthetic qualities, are essentially the same entity as when removed from the parent animal. The pelts are received either green salted or dry salted in railroad cars or auto trucks from large scale meat producing points in the midwest, far west, or Atlantic seaboard. The pelts contain large amounts of animal grease and interfibrillary water soluble proteinaceous compounds in the form of glycomucins and the like, within the skin substance, as well as large quantities of lanolin, wool grease, and animal soil attached to or entrapped in the wool. These components are removed early in the processing procedure during washing operations, the grease and lanolin being coaxed into dispersal through the use of strong detergents and emulsifiers.

The wastewater treatment system selected for the Winchester Tannery and upon which this paper is based, was chosen, of course, from a number of options. The electrochemical primary system, sometimes called LectroClear, was favored for several reasons:

1. The presence of large quantities of emulsified lanolin, wool grease, and animal fat, as well as spent fatliquor solutions, all well dispersed, dictated a clarification system which would involve flotation rather than gravity separation.
2. It was clearly demonstrated on a laboratory scale that the floated skimmings would have a much higher solids content, perhaps on the order of 10X, than a gravity system could deliver, and thus sludge storage, handling and dewatering would be expedited.
3. The continued flotation effect provided by the electrodes in the flotation basin seemed to be in the direction of maximizing primary clarification.
4. The electrolytic generation of chlorine coincident with the other products of elecrolysis appeared to be beneficial side effect that could be expected to provide some disinfection.
5. At the time of selection it was thought that reduction of ammonia was occurring within the LectroClear system, and this was a possible plus. Later this was found to be untrue.

The secondary system was likewise selected from a number of possible choices. The CARROUSEL concept was brought to our attention by E.P.A. and leather industry representatives who visited Holland in 1974. They reported rather enthusiastically as to the simplicity of design, low cost, and minimum land requirement, as well as adaptability to northern winter climate operation, ease of control of dissolved oxygen, and low operating cost. In addition, and most important, it was claimed to have the capability of not only nitrifying, but denitrifying as well. Because all of these factors seemed to indicate superiority over other known systems consultant help was secured and the decision made to install a CARROUSEL unit.

The third essential choice involved the sludge dewatering device. Since land area on the tannery property is limited it was obvious that the solid waste from the treatment plant would have to be deposited at the regional solid waste management facility. Samples were submitted. The material was accepted with the stipulation that the dry solids content would have to consistently reach 35-40%, preferably 40. Since we were aware of only one device which could reliably be expected to provide this performance, it was not difficult to recognize the need for a filter press.

The choices made in assembling this treatment plant have proved to be wise. Consistent high degree removals of pollutants have been achieved, in most cases well in excess of discharge permit requirements. Ensuing sections will go into this detail.

Characterization of Process Discharge

Shearling processing is a complex procedure using a much greater water to hide ratio than most tanneries. Since the wool is retained, and it is desired to keep the wool fibres attached to the pelts free from interweaving and tangling, the practice has developed to "swim" the skins in chemical solutions rather than limit the chemical floats to the smallest possible amount, as has become the practice in other categories of tanning operations. The liquor to skin ratio, by weight, for each fill and drain, is on the order of 2.5 to 1. With side leather the ratio is apt to be 1 to 1, and in some individual instances may be as little as 0.25 to 1. The Winchester tannery uses approximately 90 gallons of water per pelt, 7500 gals. per 1000 lbs. of green salted weight as received, or, on a weight basis, a ratio of 60 to 1. This seems very high but is necessitated primarily by the extremely soiled condition of the pelts au naturel. The tannery processes some 3000 skins per day and discharges just under 300,000 gallons of wastewater per day.

All of the above operations are carried on at the ground floor level of the tannery, and the liquid contents of the paddle vats discharge by gravity to in-floor drains and sewers. This means that it is possible for a number of vats to be discharging dissimilar solutions to the wastewater collection system at the same time. Generally the soak waters are the first to be sewered in the workday, beginning about 3 a.m. and lasting until 3 p.m., the pickle liquors are dropped from about 11 p.m. to 11 a.m., tan liquors from 7 a.m. to 2 p.m., and color-fatliquor solutions from 7 a.m. to 3 p.m. This accounts for the equalizing tank at the front end of the wastewater treatment works, where not only the dissimilar solutions are blended, but surges in hydraulic flows are absorbed.

The blended waste stream is thus a complex mixture of organic and inorganic chemicals, tanning materials, both mineral and vegetable, mineral, animal and vegetable oils, both raw and solublized, and a spectrum of dyes. It is a murky brew at best, sometimes red, sometimes blue, usually dirty gray, but always bad, and a challenge to any sanitary engineer. A typical analysis of a composite sample from the equalizing holding tank is as follows:

	mg/l
Suspended Solids	1295
B.O.D.$_5$	840
NH_4-N	32
T. K. N.	78
F. O. G.	443
Cr.	109

While this analysis may not appear to represent contamination loads encountered at chrome side tanneries, it is not as different as one might expect.

Later in this report the subject of application of the treatment process described herein to chrome side production and veg tan effluent will be discussed.

Operation of the Primary Section:

The plant obtains about 90% of its water for processing from the Ashuelot River. Drinking water and certain limited plant processing steps use city water. Dumping of the water used for processing the pelts occurs between the hours of 3 a.m. and 7 p.m. A peak in hydraulic wastewater flow occurs at about 11:00 a.m. As the wastewater leaves the plant it is passed through a stainless steel cylindrical Stahling screen where some of the wool fiber is removed. From this point the water passes into a 3000-gallon pit and it is then pumped directly into the 160,000-gallon holding tank. Wastewater in the holding tank is lifted by an immersion pump and passes through a head box which provides the hydraulic head to feed the primary Lectroclear system. pH of the wastewater in the holding tank varies between 5.1 and 7.2. The head box is approximately 15 feet above the LectroClear operation. This hydraulic head provides a flow of about 300 gallons per minute and the wastewater flows by gravity from the headbox to and through the entire primary phase.

On leaving the head box (in a 10-inch pipe), 1000 mg/l of alum is added to the wastewater. At a distance of approximately 20 feet from the alum addition and just prior to the dispersed air device, 800 ppm of hydrated lime is added from a 10 weight percent lime slurry. Both chemical additions are added by metering pump manually set to a predetermined feed. The wastewater then passes through the dispersed air device, and the vortex coagulation cell for a dwell time of 2.2 minutes after which 12 mg/l of polyelectrolyte is added (X-400 Swift anionic polyacrylic acrylamide). At this point the pH is consistently between 7.5 and 8.5. pH is monitored frequently and deviations from the just-above-neutral zone are correctly by adjustment to the lime feeding mechanism.

It must be said here that it was and is the intention to control pH by automatic adjustment to lime feeding. Equipment was provided for this at the outset but thus far manual adjustment has not only been found to be adequate, but more reliable.

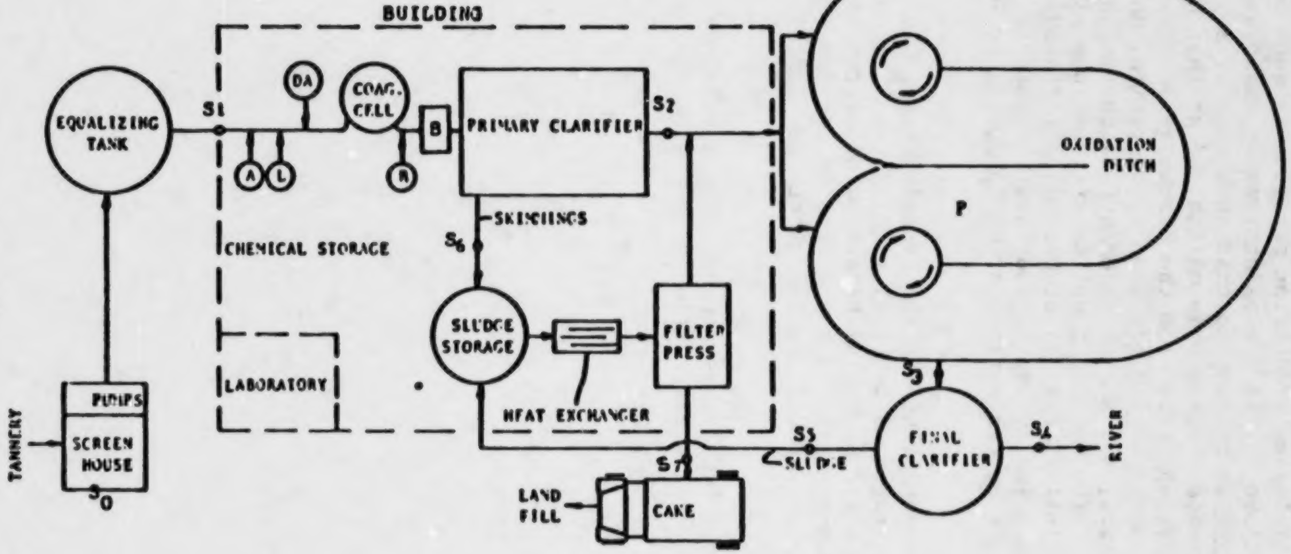
The system is designed as an "on-off" operation. This "on-off" control is carried out by a float valve in the holding tank. When the water in the holding tank is above a predetermined level, the float valve keeps all pumps and power on, and conversely when off, all off.

Secondary:

The secondary, or biological reduction section, is much more simple in day to day operation. Close checks are maintained on dissolved oxygen at selected points in the CARROUSEL raceway so that an excess of oxygen exists throughout most of the distance. Balance is maintained so that a rather short anoxic area exists just upstream from the discharge weir. Since the system is receiving on the order of 250 g/m when in operation, and the Carrousel contains 380000 gallons, the amount added and discharged during any given minute is very small. The suspended solids are continuously returned to the CARROUSEL from the final clarifier, or wasted to the sludge holding tanks for a period each day, usually two to four hours. This is required to maintain the suspended solids concentration in the CARROUSEL at 7000 to 8000 mg/l.

B.O.D. removal is accomplished in the aerobic zones. So is nitrification, or conversion of nitrogenous material to nitrate. The purpose of the anoxic section is to provide for bacterial breakdown of nitrate to nitrogen, which is released to the atmosphere. Odor problems do not develop because of the stable sludge condition, as well as the short time period in the anoxic zone at any one time.

The final clarifier accomplishes removal of suspended solids and allows discharge of a clear, colorless, effluent to the receiving stream.



KEY

A - ALUM ADDITION
L - LIME ADDITION
DA - DISPERSED AIR GENERATOR
B - BUBBLE CLASSIFIER
S - SAMPLING POINTS
R - FLOC RESIN ADDITION
P - PHOSPHORUS ADDITION

Figure 1. Schematic of the entire treatment system.

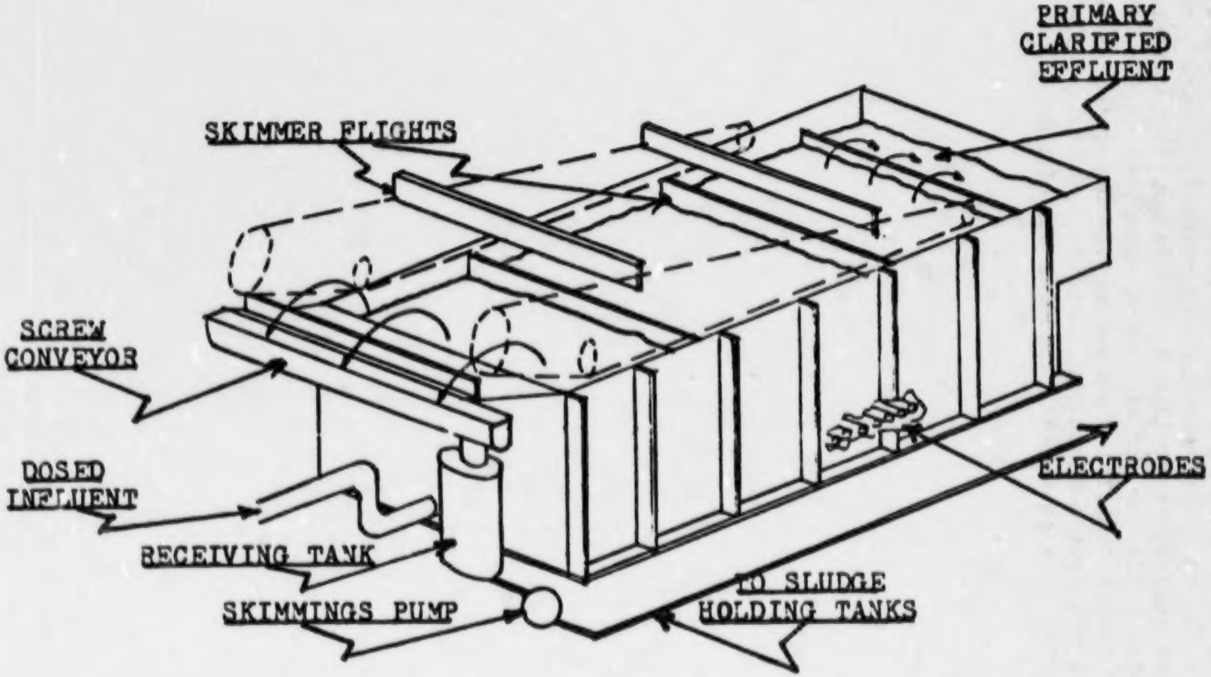


Figure 2. Schematic of the electroflotation basin.

Results:

The table of analytical figures which follows speaks largely for itself. The degrees of removal of pollutants are given in the two bottom lines, primary and secondary. It is quite apparent after reviewing these results that this system is particularly effective in reducing polluted tannery wastewater to acceptable condition for surface disposition into a river or stream. Particularly please note the high degree of conversion of soluble nitrogenous compounds, not only ammonia, but soluble proteinaceous material into free nitrogen, which escapes as such into the atmosphere.

TABLE 1. POLLUTANT REMOVALS

Date	Susp Solids*				B.O.D.5 Day*			NH-4-N*		TK-N*			NO_2*	NO_3*	FOG*			TOTAL CR*		
	S_1	S_2	S_3	S_4	S_1	S_2	S_4	S_1	S_4	S_1	S_2	S_4	S_4	S_4	S_1	S_2	S_4	S_1	S_2	S_4
10/3/77	972	308	10470	80	600	192	3	31	0.7	71	40	3.6	.02	61	1563	134	6.6	100	17.5	1.1
10/4	954	180	10390	58	690	246	3	34	0.4	70	35	3.1	.01	53	439	104	2.8	85	6.5	0.8
10/5	732	228	8888	31	683	248	3	32	0.9	73	46	4.8	1.17	48	370	43	2.2	94	11.5	0.9
10/6	1078	236	10060	32	990	312	5	24	0.4	65	20	2	.01	22	452	25	2.8	100	12	0.6
10/7	676	118	9730	32	610	216	4	37	0.6	43	28	5	.76	3	265	5	0.8	100	3.5	0.2
10/11	1022	332	9850	60	870	300	4	34	0.5	79	19	2.8	.11	13	377	21	1.2	85	12	0.4
10/12	1104	410	9730	64	870	366	7	39	1.4	76	52	4.5	.64	9	454	82	0.8	95	28	0.4
10/13	1005	412	8669	13	790	347	6	40	2.7	81	56	5.6	.30	10	480	101	18	110	39	0.7
10/14	1004	252	10110	40	830	288	4	39	3.5	83	47	7.8	.00	8	415	9	2.8	100	6	0.5
10/15	608	316	9700	54	580	312	6	34	3.8	44	27	8.7	.30	5	218	61	1.8	110	27	0.8
10/18	1164	142	10620	66	720	252	3	10	0.9	62	32	3.9	.34	23	413	5	4.8	65	1.6	1.0
10/19	1080	96	8677	44	881	286	8	36	2.4	80	43	6.7	.00	9	542	24	7	104	4.7	1.5
* Milligrams Per Liter																				
Ave.	945	252	9741	48	759	280	5	32	1.5	69	37	4.8	.30	22	499	51	7.9	96	14.1	0.7
Removal Pri.	73.4					63.1					46.3					89.7			85.2	
Tot.				94.9			99.4		95.3			98.0					98.4			99.3
Discharge Permit Requirements				50			50		1.5								15			0.5

Typical Tannery Effluent Analyses

Parameter	Side Leather Category 1 South Paris, Main	Sole Leather Category 3 Hazelwood, N. C.	Shearling Category 7 Winchester, N. H.
B.O.D. 5	1630	686	840
Suspended Solids	2718	1080	1295
Total Solids	5620	5314	14000
Fats, Oils, Greases	580	201	443
pH	10.9	9.9	5.1
Chromium-Cr	187	-	109
NHz-N	14	73	32
TKN-N	126	179	78

All values - mg/l

Examination of this table shows that there is a remarkable similarity in the nature of the wastewater from each. The parameters are not the same, of course, but the significant differences are on the order, for the most part of about 2X. Total volume is considerably greater for the side leather tannery since this is a function of sheer bulk, but it seems remarkable that the pollution level in the wastewater from each is so close, gallon for gallon.

It seems in order then, to take the stance that this described system is in fact suitable for any tannery. We would add one recommendation, however. The primary clarification system is based upon flotation of solids. It seems reasonable to suggest that it would be most efficient to remove the heavy suspended solids usually present in cattlehide tannery wastewater by providing bottom sweeps and sludge collection cavities in the surge tank. This would not add greatly to the original cost, and would remove the heavy solids at the most advantageous point.

With this recommendation we can say that a wastewater treatment plant, based upon the design principles, primary and secondary, as described herein, carefully engineered and operated, would appear to be capable of accomplishing the same removal and reduction of all parameters, and discharge a treated effluent to a receiving stream, or whatever destination with about the same degree of efficiency.

The Application of the System Described Herein to Chrome - Cattlehide and Vegetable Cattlehide Tanneries

In Section I 'Introduction' to this report it is pointed out that tanneries have been classified into seven categories to better understand the problems associated with each. Three of these categories relate to cattlehide tanneries full scale, meaning that hair removal, tanning, coloring, and fatliquoring, and finishing are involved in each.
These are:

Category 1. Cattlehide - Pulped Hair - Chrome Tan
2. Cattlehide - Save Hair - Chrome Tan
3. Cattlehide - Non Chrome Tan

Very few, if any, cattlehide tanneries save hair and chrome tan. More and more the mode has become one of soak, wash, and hair burn using strong sodium sulfide liquors, followed by reliming, and consolidated bate-pickle-tan in rapid sequence. Most tanners operating this way reclaim sulfide liquors using one of two or three methods, and all use some system to separate pulped hair solids from the waste stream before discharging to the sewage treatment system.

In a different, but similar manner, all vegetable tanners of real size, employ a hair save process. This system uses much less sulfide and produces a valuable by-product in the form of cattle hair, which has seen a resurgence in value and is now bringing almost as much per pound as it cost when purchased on the hide.

The net result of these on-plant activities has been to reduce high potential waste liquors to levels which are not so far distant as one might expect from those encountered at the subject shearling tannery. It has been stated that a shearling tannery is not a typical tannery since it is neither cattlehide, nor does it have a beamhouse, and therefore wastewater treatment technology transfer is not possible. Since A. C. Lawrence operates tanneries classified in categories 1, 3, and 7, and company technicians are familiar with the details of all of the other categories, they have the capability of recognizing the dissimilarities and parallels. Careful scrutiny divulges that there is not all that much difference.

In order to rather definitely establish the similarity we present the following table of typical analyses of wastewater from our own tanneries, all of which we consider to be more or less representative of complete tanning operations in their respective categories.

The 1977 "Mid Course Corrections" to the Federal Clean Water Program

by

Joan Faier and Michael Cook

Recent legislative changes in the United States' water pollution control program are of interest to Japan and other countries because they reflect changing views on what constitutes sound planning, appropriate technology, balanced institutional relationships, and acceptable procedures for resource allocation and governmental regulation of the private sector.

Last December, the U.S. Congress passed the Clean Water Act of 1977 (P.L. 95-217), amending the Federal Water Pollution Control Act of 1972 (P.L. 92-500). The 1972 law changed the course of Federal water pollution control but the 1977 legislation only provided adjustments, usually referred to as "mid-course corrections", to the ongoing program.

The authors of the 1972 Act expected that these adjustments would be essential. They provided for the National Commission on Water Quality to review implementation of P.L. 92-500. The Commission's recommendations, coupled with recommendations from EPA and some independent Congressional initiatives, formed the basis for the 1977 amendments.

More than half of the 100 amendments enacted in 1977 impacted the grants program for construction of municipal wastewater treatment facilities. Despite these numerous corrections, however, the basic program remains essentially intact. We estimate, for example, that the many new provisions only increase the Federal and State resources necessary to manage an optimum grants program by about five percent.

The 1977 Act authorized $24.5 billion for the Construction Grants Program in the five years beginning in FY 1978. This and other major authorizations for other programs came on top of our $18 billion obligated in the grants program since 1972 (including $6.7 billion during FY 1977).

Passage of the Clean Water Act of 1977 demonstrates the continuing powerful public endorsement for cleaning up the nation's waters. This public support has also been demonstrated recently at the State and local level. For example, during the recent, widely-

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publicized California referendum on Proposition 13, while taxpayers voted for spending less government money on items such as social welfare programs and education, they simultaneously voted for devoting more government money to the solution of water pollution problems. The bond issue passed on the same ballot in California provides $350 million over a five year period for matching State funds. In the face of the growing citizen furor for small government, this citizen support of water pollution expenditures was not highly publicized. But it may reflect a telling trend.

The current high rate of expenditure on water pollution control in the United States comes on a foundation of over a century of continuing, though often limited, investment in needed sewers and treatment facilities. Japan's recent major investments in water pollution control are in many communities providing the first sanitary sewers. I would anticipate, if our experience is a guide, that Japanese public support for a large construction program will continue unabated in the foreseeable future.

The 1977 Clean Water Act introduced some totally new provisions into the law, shifted the focus of some of the old provisions, and reemphasized certain aspects of the old law, formulated in 1972. One of the most salient shifts in emphasis as a result of the new law is Congress' active intent to encourage innovative and alternative (I/A) technologies. Second, the new law shifted the water pollution control program's focus towards a better balance between urban and rural areas and towards consideration of the special problems that small communities must face. Third, the new law also provides a greater role to the States in making certain administrative decisions--including management of their priority lists and their construction grants programs. Fourth, the Act enhanced the flexibility of programs for control of conventional pollutants, while increasing the emphasis on control of toxic pollutants. Fifth and last, the law adjusted some deadlines and requirements to impose more reasonable demands at the local level.

One of the most crucial shifts in the Construction Grants Program is the active preference for I/A technology that Congress incorporated into the 1977 amendments.

Under the new Act, municipal grantees must demonstrate that they considered all feasible I/A alternatives when preparing their facility plans.

The 1977 amendments reinforce the 1972 Act's intent to encourage the use of I/A technologies by providing concrete financial incentives to grantees who use I/A technologies, even if the technologies are not the least costly ones. The 1977 amendments provide a 15 percent cost preference and 85 percent rather than 75 percent Federal grants for I/A technologies.

The Federal government also will pay 100 percent of the costs for repairs or modifications of an I/A system which fails to meet its design standards at reasonable cost. EPA decided that this repair option applies for two years after final inspection. The 1977 legislation even allows States to readjust their priority list to encourage I/A technology.

The preference for I/A techniques has its sources in several places. In an environmental message to Congress in the fall of 1977, President Carter voiced his support for the design and construction of cost-effective, publicly-owned wastewater treatment facilities that encourage water conservation as well as adequately treated wastewater. Congress expressed its intent, both in 1972 and in the current law, to focus efforts particularly on wastewater treatment processes which renovate and reuse wastewater, as well as recycling the organic matter and nutrients, and recovering energy in a beneficial way. Land treatment of wastewater and land spreading of sludge exemplify processes which achieve these objectives.

The proposition underlying these new amendments is that substantial environmental benefits should result from reliance on I/A technology. These potential benefits include a more expedient use of resources, energy conservation, cost savings, and generally more efficient methods of wastewater treatment. I/A technologies should improve some of the inherent drawbacks to conventional systems, such as the production of very large quantities of sludge which then must be disposed of; pollution from pollutants such as oxygen demanding materials and nutrients; high operating costs; waste of precious water; and excessive energy demands. But some of these same problems may still plague a particular I/A method.

One of the largest controversies surrounding I/A policy is how to define "innovative" and "alternative". EPA headquarters and our office in Cinncinati, the Environmental Research Information Center (ERIC), will play a key role in determining an appropriate definition for I/A. We are currently developing guidance to help the Regions, because they will eventually make decisions on whether an individual project is I/A on a case-by-case basis.

Beyond these considerations of a broad definition of I/A, Congress and EPA have identified specific processes or systems as I/A. For instance, land spreading of sludge is one kind of preferable I/A technology. Land spreading techniques are based on the premise that sludge can be used both creatively and productively. Sludge can be used as a soil conditioner for crop lands, while conserving and recycling organic matter, nitrogen, phosphorus, and certain essential trace elements. Another use of stabilized sludge is for land application to enhance parks and forests and reclaim poor or damaged terrain. Environmental organizations are currently studying the possibility of using sludge for reclamation of strip-mined land. We believe that we have sufficient knowledge to implement safely those systems, but we are continuing to study the extent to which this practice could result in entry of toxic substances into the human food chain and pose a public health problem. We are also establishing programs to ensure good management of sludge spreading operations to minimize the possibility of adverse health effects.

One of the major problems in promoting the use of I/A under the old Act had been the engineering community's strong tendency to continue to use tried and true conventional techniques and processes with which they were most familiar. Because of the potential risks involved, those who favored I/A technologies found difficulty in gaining acceptance for them. Prior to the passage of the new Act, grantees did not even have any financial incentive to turn to I/A if their conventional systems were working at least reasonably well. Conventional systems had two advantages in this regard--they already had public acceptance and a proven capability to meet effluent standards at a known cost.

On the other hand, the engineering community felt that I/A processes had the potential to be more trouble than they were potentially worth. For example, specialized manpower might be required and would cost both additional time and expense to train. Reaching the citizenry with information about I/A in order to generate public support also promised to be another financial constraint and a large commitment of time and energy. Furthermore, grantees found it difficult to judge what an I/A system would cost because less operating and cost data for new technology were available to do a reliable cost-effective comparison with conventional systems.

The new law will help overcome some of the old obstacles to I/A technologies. EPA wants to avoid potential new problems, such as a shift from the extreme of little or no consideration of I/A to the other extreme, an endless, proliferating series of unnecessary studies of new methods. Also, avoidance of the disruption of design and construction work begun before the passage of the 1977 Act is crucial.

Along with an analysis of I/A alternatives during the facility planning period, the municipal grantee must also include an analysis of potential recreational and open space uses in his facility plan. This required analysis provides a community with a chance to maximize its environmental benefits at relatively little extra cost. Multiple use facilities, combining a wastewater treatment plant and a recreational facility, have the advantage of easy access to clean water. The stipulation that grantees must consider recreation and open space uses in their facility plan does not apply retroactively for facility planning already begun prior to the passage of the 1977 Act.

Japanese experience will be particularly useful to us in this area. Many Japanese treatment plants have been covered over and enhanced by formal parks and other kinds of recreational uses in order to make them more acceptable to neighboring residents. Although Japan and the United States both have an interest in the concept of multiple use facilities, the two countries have approached the same goal from different perspectives. Due to Japan's dense population, the only available site for a wastewater treatment plant was often close to a residential area. The Japanese confronted the problem of winning needed citizen support for their choice of location. I understand that combining a sewage treatment plant with a park has often been the pivotal element which tipped the balance towards community acceptance. In the United States, we have had fewer major problems with choosing sites in the past, although they are occurring now with greater frequency. Moreover, a growing concern about conservation and the environment has spurred among environmental planners the popularity of the concept of a multiple use facility. Planners have begun to view the multiple use facility as the most cost-effective answer for communities which desperately want a new recreational facility but which are also experiencing a financial squeeze.

The municipal Construction Grants Program's underlying philosophy and primary objective is to clean up the water. Therefore, while the municipal Construction Grants Program does not provide any money for the design and the construction of recreational facilities, the availability of funds through 201 to at least analyze recreational uses is an inviting incentive for planning multiple use facilities.

EPA is currently developing guidance for recreational use funding. We are particularly working with the Department of Interior's Heritage Conservation Resource Service (HCRS) in order to help grantees secure funding for multiple use projects. With the National Parks and Recreation Association (NPRA), we are also developing a handbook which will give grantees suggestions for multiple use projects and for funding alternatives.

The concept of combining parks and sewage facilities has met some public resistance but many notable successes exist also. One such success story occurred in Naperville, Illinois, where abandoned sewage treatment plants were converted into parks. The city constructed a new regional wastewater treatment plant and abandoned four small ones along the west bank of the Dupage River. The abandoned plant sites fit conveniently into the Naperville Park District's plan to establish a greenway along the river. The Naperville Park District has also leased buffer lands for recreational use around the new regional wastewater treatment plant. The district provides community gardening plots on this land and is constructing a canoe access site on a portion of the land abutting the west branch of the Dupage River.

Another unusual opportunity to use recreational space arose in the Chicago area. The Metropolitan Sanitary District of Greater Chicago, which treats most of the wastewater in Cook County, operates a system of three canals in the Chicago area for shipping purposes.

As the Sanitary District improves its collection and treatment operations, pollution of the canals is decreasing and vacant land along the canals, which the sanitary district owns, is becoming more desirable for recreation. For example, the City of Evanston has located an ecology center and arboretum on the Sanitary District's land and the Chicago Park District has developed recreational facilities on some canal lands inside the City of Chicago. In addition to planned recreational facilities and parks, the concept of leaving open space around the sewage treatment plant has proven to be even more palatable to citizens. The preservation of natural habitat for wildlife is one such use.

EPA revised its cost-effectiveness guidelines in accordance with the new law in order to provide for cost-effective sizes and sufficient reserve capacity for wastewater treatment works while at the same time to avoid overdesign problems. Coverage under the new guidelines included guidance and alternative procedures for forecasting growth of population, for estimating wastewater flows, for determining cost-effective construction staging periods, and for providing extra capacity beyond that determined to be cost-effective.

The cost-effective analysis is designed to include an evaluation of non-monetary factors, including primary and secondary environmental effects, implementation capability, use and recovery of energy and scarce resources, recycling of nutrients, operability, performance reliability, and flexibility. The most cost-effective alternative will be the wastewater treatment management system determined from the analysis to have the lowest present worth and/or equivalent annual value without overriding non-monetary costs and to realize at least identical minimum benefits in terms of applicable effluent limitations. Thus, the cost-effectiveness analysis is more complex than merely finding the cheapest facility plan alternative.

In keeping with the President's policy for conservation of water and our cost-effectiveness guidelines, which also demonstrate a preference for energy-efficient, conservation-oriented facilities, we are instituting a program to educate the public in water conservation and reuse/recycling. Section 214 of the amended Act mandates citizen involvement and awareness in the national water effort. EPA is required to set up a program of "public education on recycling and reuse of wastewater (including sludge), the use of land treatment, and methods for reduction of wastewater volume by December 27, 1978." Our program is currently underway, enlisting the aid of various environmentally-concerned groups, and will be implemented on schedule. I understand that Japan is also moving toward greater public participation in planning of wastewater treatment plants, although the options for participation are not as formalized as in the United States.

The emphasis on I/A technology, improved cost-effectiveness analysis and public participation are all part of the major new emphasis in the grants program on improved planning which gives careful consideration to options appropriate to each local situation rather than traditional solutions and rules of thumb which may prove costly and less beneficial.

A second major shift in focus in the 1977 Act is the emphasis on meeting the needs of small communities. In the past, Congress felt that the grants program was overburdening small communities both economically and administratively. Therefore, a common theme running throughout the 1977 amendments is that the small communities should receive more assistance and flexibility and should be subject to simplified paperwork requirements.

EPA experience paralleled Congressional findings on the need to tailor the grants program to local needs. Projects in small communities which cost hundreds of dollars per household per year in local debt retirement and operation and maintenance expenditures are not unusual. Financial programs imposing front-foot assessments of thousands of dollars have unfortunately been set up to provide the local share in some communities. EPA has even seen some cases where the cost of the wastewater treatment facility exceeds the entire assessed value of a community.

The new amendments provide small communities with a combination of incentives, requirements, and supplemental assistance. One provision is a four percent set aside from allotments of rural States for alternative treatment systems in towns of 3,500 or less or dispersed areas of larger communities. These funds must be used in rural States or be lost.

Alternatives to conventional treatment systems for small communities are defined as systems other than the conventional gravity sewers leading into centralized treatment systems. They include: septic tanks or aerobic treatment units with soil absorption fields; various add-ons to septic tanks, including mound systems, aerobic units, low-water or no-water toilets with greywater treatment systems; and innovative sewers, including both vacuum and pressure sewers.

A second related provision makes grant eligible privately-owned individual systems serving one or more existing residences or small commercial enterprises. A public body has to apply on the behalf of a number of individual units to receive a grant for such a system. Additional stipulations include: a user charge system must be installed to recover operation and maintenance costs; commercial enterprises must pay back the Federal share eventually; and the system must be less costly than a conventional system.

The provision of the law which increases the Federal grant from 75 percent to 85 percent of the eligible costs for alternative or innovative systems is particularly helpful to small communities. It provides a larger Federal share to communities which frequently are experiencing higher per capita required new investment costs than more populous communities.

A fourth provision authorizes combined Step 2+3 grants for projects up to $2 million (up to $3 million in high cost States) in towns of 25,000 or less. This provision will cut down on the amount of paperwork in small communities and will save money by accelerating the construction schedule.

A fifth provision authorizes EPA to make a grant to States of up to two percent of the State allocation or $400,000, whichever is greater, in order to assist with management of the Construction Grants Program. In addition, these funds may be used to help small communities learn how to manage their construction grants program.

Finally, a sixth provision authorizes EPA to provide technical and legal assistance at the request of the grantee to help administer or to enforce any contract related to the program. The goal of all these new provisions is to ensure that the scope of the facility plan is tailored to the nature and extent of local problems.

A vivid example of the kind of problem that these provisions are designed to prevent occurred recently. A small community prepared a facility plan which was approved, and a Step 3 grant was awarded to build new sewers and an oversized treatment plant. The plan called for payback of the huge loan for the local share in only six years. These loan payments were to be met largely by front-foot assessments on property owners. The result was astronomical assessments: one property owner's share skyrocketed to $80,000. Operation and maintenance of the facility was also expected to be very costly. Because the citizens rebelled, although the contract for construction was executed, work never began. In the meantime, EPA made several recommendations: to reduce the project to a more reasonable size; seek assistance from the Farmers Home Administration (FmHA) for the local share; refinance the project over a much longer period; and meet the debt retirement costs by a more reasonable financial plan. With State assistance, this near-financial disaster could have been averted much earlier.

Under the new law, States can use a portion of the State construction management assistance grants to pay 100 percent of the costs of providing technical and managerial expertise to small communities.

A third major area in which political forces shaped the 1977 amendments grew from a need to clarify relationships and to allocate resources between Federal and State control of programs. Implementation of the 1972 Act was fraught with conflicting interpretations of the division of authority and responsibilities between State and Federal agencies. Critics claimed that the Federal government was usurping State power and that overlap plagued Federal and State programs. The 1977 amendments, at least at first glance, attempt to concentrate more responsibility in the States--through provisions such as construction management assistance grants (CMAG) and State management of priority lists. However, this seeming realignment of responsibility from Federal to State government may be deceptive. While the States are taking on more responsibility in the daily management of the program, they may be in fact working within a more constricted Federal framework, in which the Federal government sets the goals and standards even more strictly under the more detailed requirements of the new Act.

Nevertheless, one of the goals of the 1977 amendments is to decentralize the management of the Construction Grants Program. The Federal government has been performing an increasingly important management function since the grants program began in 1956, so this 1977 shift represents a major operational change. The challenge created by this change is how to maintain the uniform, high national quality of the program while simultaneously subjecting applicants and grantees to one less layer of bureaucratic action. EPA has strongly supported this provision because it can potentially bring a large net increase to the resources available to administer the Construction Grants Program. These resources would be hired by the States with Federal monies.

The amended section 205 authorizes EPA to make funds available to States to manage their construction grants programs and, as needed, to hire qualified staff to provide training necessary to carry out effectively the delegated functions. The legislative history provides that CMAG is the number one priority for use of section 205(g) funds, although funds may also be used for management of the permit programs and the water quality management programs.

The award of a State construction management assistance grant takes place after the execution of a delegation agreement between the State and EPA. The delegation agreement is the mechanism for implementation of the construction grants management program and shall address both the functions to be delegated and the State's capability to conduct these functions.

The most current statistics on the status of the delegation agreements reveal that California, Illinois, and Texas have reached partial or full delegation. By October 1, 1978, another six States--New Hampshire, Georgia, New York, Wisconsin, Alaska, and Idaho--should also reach full delegation. By January 1, 1979, EPA expects that nine additional States--Connecticut, Vermont, Virginia, South Carolina, Michigan, Minnesota, Colorado, Montana, Maryland--will also meet full delegation requirements.

In order to assure continuity from one fiscal year to the next, one of EPA's major concerns in State programs is that funds obligated under CMAG shall be available for expenditure for a period of up to five years. The grant may be amended annually as subsequent allotments are appropriated by Congress. EPA believes that States will have sufficient funding stability to hire, train and maintain qualified staff under this provision even if the timing or amount of future Congressional appropriations vary.

The regulations also provide for public participation in the delegation process during the development, review and approval of a new delegation agreement or modification of an existing agreement and require a public hearing to be conducted on a draft delegation agreement whenever the Regional Administrator determines that significant public interest in such a hearing exists. While many items are delegatable to the States, the following duties are not: grant award and amendments, decisions on conformance with the National Environmental Policy Act, compliance determinations on civil rights issues, bid protests, disputes, and review of audit exceptions.

Another area in which the new law gives the States more authority is in setting State priority lists for building their wastewater treatment facilities. The new Act gives the States exclusive authority to rank the categories for their projects. The categories in the 1977 Act are those relied on in the survey of the cost of needed publicly-owned treatment works (Needs Survey). The State now has the sole authority to weight this list towards any particular category it finds necessary.

Under the old law, EPA regional offices used to review an entire State priority list and could direct the State to redo the entire list if they disagreed with it. As a result of the new law, the EPA regional offices can only look at a particular project and can decide on a case-by-case basis whether that project meets the "enforceable requirements" of the Act. If not, EPA can remove projects on the State list which do not meet "enforceable requirements" and alternate projects must be added to use available funds. But under the 1977 law, EPA cannot question the States' choices of overall emphasis in their lists, so long as projects are designed to obtain pollution control objectives. The legislative history of the 1977 Act indicates that State priority list planning and management should be closely linked to meeting unfulfilled treatment needs before other eligible needs can be funded.

The overall result of the new amendment on priority lists is that national control over resource allocation is more restricted than before. These restrictions contrast with centralized resource allocation in Japan.

The 1977 Act also made substantial changes in EPA's role in the section 404 dredge-and-fill permit program, which the Corps of Engineers had exclusively operated under the old law. Under the new amendments, the States may establish through their own 404 or water quality management program, permit programs which would replace the existing Corps permit program, and EPA then takes on the role of approving and overseeing them. The delegation to the States is advantageous because States can more easily customize their programs according to the areas' conditions. In order to clarify that non-point sources of pollution, such as agricultural run-off, are not intended to be a part of the 404 program, the Act also stipulates explicit exemptions from the program. Finally, the new Act extended the 208 planning period from two to three years, and provided an additional funding authorization.

The fourth area of emphasis in the new Act is a new stress on control of toxic pollutants coupled with enhanced flexibility in control of conventional pollutants. The 1977 Act reflects a new commitment to set standards for toxic pollutants as well as to develop techniques to pretreat chemical pollutants discharged by industries. Our interest in toxic pollutant control has been growing rapidly, and this is another area in which Japanese expertise would be invaluable because of extensive work with toxics.

The 1977 Act addresses pretreatment standards for industrial pollutants which pass through or interfere with municipal treatment processes or which contaminate sewage sludge. EPA is authorized to require the development of a local pretreatment program, which establishes limits for specific pollutants contained in industrial discharges, in order to enforce national toxic pretreatment standards. EPA is also authorized to provide for compliance monitoring and to assure adequate revenue to run a local pretreatment enforcement program.

EPA also has the Congressional mandate to establish national toxic pollutant standards, based on the best available technology (BAT) economically achievable and the more stringent requirements of section 307(a). A Court Order based on a suit by the Natural Resources Defense Council, (NRDC Consent Decree), mandated that effluent limitations for 21 major industrial categories based on BAT be established for 65 specified toxic pollutants by December 31, 1979.

In 1977, Congress rewrote the toxic pollutants portion of the Federal Water Pollution Control Act, codifying the NRDC Court Decree--although extending the court-imposed deadlines and empowering EPA to add to and to subtract from the court's list of 65 toxic pollutants. Under the new legislation, EPA has until July 1, 1980, to promulgate BAT toxic effluent guidelines, on an industry-by-industry basis, for the

Consent Decree pollutants. Industries are required to comply by July 1, 1984. For substances which are later added to the initial list, industry will have three years following promulgation of effluent guidelines to comply with them.

In a final 1977 action to control toxic and hazardous substances, Congress approved language regulating industrial management of toxic and hazardous materials which might enter the environment other than through effluent discharges. EPA is authorized to establish "best management practices" (BMP) to be implemented as provisions of National Pollutant Discharge Elimination System (NPDES) permits for the control of plant site runoff, leaks, spillages, sludge, and other waste disposal and drainage from raw material storage sites.

The BMP provision of the 1977 amendments plays a key role in the nation's defenses against toxic substances. Under the Toxic Substances Act enacted in 1976, EPA inventories chemicals produced and/or used in the United States. Then, the Clean Water Act, with the new toxic provisions of 1977, provides control over dispersal into the nation's water of those toxic substances which continue in use. Then, in turn, the Resource Conservation and Recovery Act of 1976 provides comparable control over disposition of toxic-laden sludge. The BMP provision closes a potential gap left by the various other applicable laws.

However, the new law seeks to cover a broader range of materials than those classified strictly as "toxic." Section 311 declares it a national policy to prevent discharge of oil or hazardous substances into United States' water and to establish procedures by which EPA could act to prevent or to respond to spills.

Under the old law (P.L. 92-500), Congress required that industries meet effluent limitations by 1977 which require the application of best practicable technology (BPT). Factors taken into consideration in developing these effluent limitations included: the amounts and characteristics of pollutants, the degree of effluent reduction attainable through application of BPT, and the total cost of application of technology in relation to effluent reduction benefits to be achieved. The EPA developed effluent limitations from these requirements which specified control primarily of biological oxygen demand (BOD), suspended solids (SS) and acidity (pH); these were the pollutants traditionally controlled by municipal systems.

The 1972 law also required the application of a higher level of control, best available technology economically achievable (BAT), based on similar but more stringent requirements and to be achieved in 1983. Initially these requirements were interpreted as simply more stringent limitations on the same pollutants covered by BPT. Industry objected to this on the grounds that (1) BPT would result in the majority of the effluent reduction required of these pollutants (an approximate 85 percent removal average across industry) and (2) that to remove the remaining 10-15 percent of these pollutants would cost approximately as much as removing the first 85-90 percent.

Separate but related actions by EPA, environmental groups and Congress have resulted in a compromise solution to this problem. First, the NRDC Consent Decree mentioned earlier was jointly developed by EPA and the environmental groups to shift emphasis to the control of toxic pollutants. As a corollary to this action, the Agency initiated a review of BAT limitations for non-toxic pollutants (e.g., BOD, SS, pH) to be sure they were "reasonable;" if they were found unreasonable they would be revised.

Congress accepted this general approach in the 1977 amendments. As mentioned earlier, the use of BAT to control toxics (as in the NRDC Consent Decree) was accepted with minor changes. Second, those pollutants not defined as toxic were divided into two groups, conventional and non-conventional. Conventional pollutants were defined by Congress as BOD, SS, pH, and fecal coliform plus any additions promulgated by EPA; EPA has proposed the additions of COD, oil and grease, and total phosphorus. Industrial discharges of these pollutants are now to be controlled through application of best conventional treatment technology (BCT), which is similar to BAT with a less stringent cost test. EPA is reviewing all BAT standards for these pollutants; those found "reasonable" in light of this new cost test will be retained as BCT; those found "unreasonable" will be revised (with lower and upper boundaries of BPT and BAT).

Third, non-conventional pollutants were defined as those "gray area" pollutants not defined as toxic or conventional. These pollutants are to be controlled by application of BAT, but are eligible for economic or environmental modifications (such modifications are not available for toxic and conventional pollutants).

Another controversial area is advanced wastewater treatment (AWT) for municipal wastewater treatment plants, where the issues at stake are similar to those in the BAT controversy. In addition to the removal of BOD and SS, AWT can also remove some toxic pollutants and some nutrients. However, opponents of AWT also argue that in many cases the advantages of stringent levels of treatment are small compared with the great expense of the AWT facilities.

The 1976 Needs Survey indicates that about half of the municipal facilities serving over 60 percent of the population will have to meet requirements more stringent than secondary at an incremental cost of $11 billion.

In order to improve data for AWT decision-making, EPA and the States are making fundamental changes in their water quality monitoring programs, shifting to intensive surveys in selected areas rather than expensive regular inventory at thousands of fixed stations. Regional offices must now complete a checklist before making a Step 2 and Step 3 grant for any project providing treatment more stringent than secondary. This checklist requires information about project costs and justification of the basis for effluent requirements. EPA suggests action to be taken if available information does not demonstrate the need for treatment levels more stringent then secondary.

Another of the Act's controversial aspects, which may be of particular interest to Japan, is the inclusion of a waiver from secondary treatment for marine municipal discharges. The old Act required secondary treatment for all wastewater discharged anywhere--lakes, oceans, estuaries, etc. However, the National Commission on Water Quality Study found that blanket application of secondary treatment is not necessary. Waivers for marine discharge may be cheaper and more efficient if the ocean conditions and the nature of the pollutant discharges are correct. One major factor in the determination of whether a marine discharge permit should be granted is the depth of the water, which affects the likelihood of the pollutant's easy dispersal.

The 1977 amendments provided a waiver for municipal discharges if eight criteria are met. These are: 1) applicable water quality standards for the specific pollutants involved must exist; 2) modification will not interfere with attainment or maintenance of water quality, which assures protection of public water supply and of a balanced, indigenous population of shellfish, fish, and wildlife; 3) the applicant must establish a monitoring system to record impact on a representative sample of aquatic biota; 4) modified requirements must not result in any additional requirements on any point or nonpoint source; 5) pretreatment requirements must be enforced; 6) the applicant must establish a schedule of activities designed to eliminate toxic pollutants from non-industrial sources; 7) no new or substantially increased discharges are to be allowed to occur from the point source of the pollutant to which the modification applies; and 8) funds available under Title II must be used to achieve the effluent reduction required by section 201(b) and 201(g)(2)(A) or to carry out the requirements of this amendment. In short, impacts on marine environment must be carefully considered before granting the exemption.

Waivers are only for pollutants which fall within the definition of secondary treatment. They are reviewable every five years. For the western coast of the United States and other coastal areas, such as Hawaii, Puerto Rico, and Alaska, natural factors may be sufficient to minimize the impact of traditional forms of pollution from publicly-owned treatment works. In California alone, the money necessary to achieve secondary treatment could be $1 billion--whereas some money-saving opportunities for marine discharge waivers do exist.

In addition to the 201 municipal Construction Grants Program, the Act mandates our 208 program which is designed to place municipal planning in the broader context of water quality management on a regionwide or even a statewide basis. One of the major differences between the United States and Japanese framework for pollution control is that Japan has a more comprehensive, overall plan which will only be paralleled in the United States after full implementation of our 208 water quality management planning program occurs. In Japan, the Ministry of Construction--one of the three agencies responsible for clean water--approves long-term, comprehensive sewage treatment plans which estimate costs, analyze alternatives, and outline the configuration and nature of recommended facilities. Individual five-year and three-year plans supplement the master plan by providing the details. Our 208 program is more loosely constructed.

The new law changed the 208 program's role in rural areas. Section 208(j) of the Act directs the Secretary of Agriculture to develop, with the EPA Administrator's concurrence, a national program of technical and financial assistance for rural wastewater quality management. The new Act authorized $600 million over the next several years to contract with farmers to use BMP on their land for non-point source pollution elimination. The section stipulates several provisions for implementation of this cost-sharing programs: (1) a 208 plan must be approved before a project within the area State can be approved; (2) only BMP which are certified by a designated management agency to be consistent with approved 208 plans are eligible for cost-sharing. (BMP include practices such as contour plowing which are conservation measures); (3) the Secretary of Agriculture will enter into a long-term contract, for a duration of five to ten years with individual landowners; and (4) priority will be given to those areas and sources that have the most significant effect on water quality and funds will be apportioned based on water quality effect not on a predetermined allocation formula.

The rural management program provides a major incentive to control both conventional and toxic pollutants from agricultural lands. It thus fills a major gap in the previous water pollution control program.

The last important area of change in the new law was an adjustment of deadlines and requirements to impose more reasonable demands at the local level.

Section 204 of the new Act has modified the user charge system by allowing grantees to use dedicated ad valorem tax systems for the collection of operation and maintenance costs if a portion of the grantee's ad valorem taxes were dedicated for such use in December 27, 1977. Under the old Act, charges were exclusively based on actual use.

Under the industrial cost recovery (ICR) system, municipalities are required to collect a proportionate share of the Federal contribution to wastewater treatment plant construction from industrial users and return it to the Federal government. The ICR program has been the subject of controversy, and the new Act resulted in a narrower definition of "industrial user" and an 18 month moratorium on payment of ICR to the Federal government while the EPA conducts a study evaluating the effectiveness of the program.

After passage of the old Act, treatment of conventional industrial wastes was generally on schedule but compliance by municipalities was lagging. In the 1977 amendments, Congress established a new pollutant classification system with different deadlines and treatment requirements for each class. Requirements for municipal treatment remain basically the same as in 1972, except that the 1977 treatment deadlines may be extended to no later than July 1, 1983 in some cases--where construction cannot be completed on time or where matching Federal financial assistance was not available in time to achieve compliance with the 1977 deadline.

The 1977 amendments established three categories for industrial pollutants--conventional, toxic, nonconventional ("all others")--and a general deadline of July 1, 1984. If an industrial polluter has acted in good faith but has not met the 1977 BPT deadline, instead of enforcement proceedings, EPA can extend the deadline to no later than April 1, 1979. The 1977 BPT deadline may also be extended to July 1, 1983, in cases where the industrial discharger has contracted to discharge to a publicly owned treatment system, but the public system is not yet ready to accept the discharge. Finally, the 1984 BAT deadline, for toxic and nonconventional pollutants, may be extended to July 1, 1987, to allow replacement of existing production capcity with an innovative process which will result in greater pollution reduction or lower cost, and which has potential for industry-wide application.

In short, we are pleased with broad public support for the goals of the Clean Water Act as we implement the basic changes: shifts towards innovative and alternative technologies, the focus on small rural community problems, the decentralization of the program to increase State control, an increased concern for toxic pollutants, and adjustment of requirements to impose more reasonable demands at the local level. Japanese experience in these areas will be of use to us, and hopefully our experience in implementing our new major law will be of help to you in your clean water efforts.

THE CONSTRUCTION GRANTS PROGRAM - A REGIONAL PERSPECTIVE

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ABSTRACT

An EPA Regional Office perspective of the administration of the Construction Grants Program is presented in this paper. The interdependence of Federal, State, and local governments in attaining National goals and significant programmatic issues facing all levels of government are discussed.

INTRODUCTION

The first Federal financial aid in the construction of municipal sewage treatment works began in 1956 with the passage of the Federal Water Pollution Control Act (Public Law 84-660). This Act authorized an appropriation of $50 million a year for grants to municipalities. The grants were limited to 30 percent of the eligible project cost not to exceed $250,000 on any one grant.

From this modest beginning the Municipal Wastewater Treatment Construction Grants Program has grown to become the largest source of Federal financial assistance in the U.S. The 1972 Amendments to the Water Pollution Control Act authorized $18 billion to assist municipalities in the construction of sewage treatment plants, intercepting sewers and pumping stations, collecting sewers, sewer system rehabilitation, and the correction of combined sewer overflows. Further, the Federal proportionate share was increased to 75 percent of the eligible costs for planning, designing, and constructing these facilities.

The basic Act was most recently amended in December, 1977. These Amendments extended the construction grants program and provided additional authorizations of $25.5 billion for the period 1977 through 1982.

The Amendments to the basic law have not only increased the amounts of money in the construction grants program, but also the complexity of the program has increased dramatically. From the rather simple beginning in 1956, the number of requirements that must be met to qualify for a grant has increased. Before a grant can be issued the grantee must comply with the National Pollution Discharge Elimination System (NPDES Permit), complex planning requirements, cost-effectiveness analysis, requirements of the Environmental Policy Act, and numerous administrative requirements.

Throughout all of the various Amendments discussed above, the underlying theme established by Congress for administering the construction grants program has been to create an operating structure in which all levels of government share responsibility for attaining objectives of the Act. Within this framework, the Federal, State, and local governments are each assigned primary roles for carrying out various activities of the program.

Overall program direction is the responsibility of the Environmental Protection Agency's Headquarters in Washington, D.C. This direction takes the form of regulations, program requirements, and general operating guidance. EPA's ten Regional Offices (Attachment 1) interpret and apply these directives to the specific conditions that exist in each of the States within a geographical region. The Regions approve of each State's continuous planning process which establishes water quality standards and identifies projects for funding. The Regions perform final administrative and technical review of project applications submitted by the States and award grants to qualifying municipalities.

The State Water Pollution Control Agencies have primary responsibility for identifying water pollution abatement needs, selecting projects through the continuous planning process, and delineating project service areas. States are also responsible for initial receipt and review of grant applications for Federal assistance and for ensuring these applications are consistent with the planning process, applicable State regulatory requirements, and all technical, administrative, and legal provisions of the national program.

Applications for projects to abate local water pollution problems and to assist in complying with effluent limitations are prepared by municipalities usually with the assistance of an engineering consultant. After grant assistance is provided, the municipality or grantee is responsible for the performance of the grant and any subagreements covered by the grant.

When discussing the detailed responsibilities of each level of government, it is perhaps best to subdivide the program into five major sequential components or steps. These are (a) the municipal needs survey, (b) the annual State strategy and project priority list, (c) facilities planning or Step 1 process, (d) design of treatment works or Step 2 process, and (e) construction of treatment works or Step 3 process. An EPA regional perspective of the responsibilities that each level of government has in each of these steps follows.

MUNICIPAL NEEDS SURVEY

When Congress established the goal of fishable and swimmable water by 1983, it became necessary to accurately define the cost of achieving this goal. EPA, in cooperation with the States, is required to prepare a biennial report to Congress which estimates the cost of achieving this goal Nationwide. In preparing this estimate the needs are divided into five major categories. The most recent total estimated national needs for the projected 1990 population were last reported to Congress in February, 1977, and are summarized below by category:

1976 NEEDS SURVEY SUMMARY COSTS
(January, 1976 Dollars in Billions)

Needs Category		1990 EPA Assessment ($ Billion)	1990 State Estimate ($ Billion)
I	(Secondary Treatment)	12.955	13.199
II	(More Stringent Treatment)	21.279	22.049
IIIA	(Infiltration/Inflow)	3.017	3.765
IIIB	(Replacement and/or Rehabilitation)	5.486	5.732
IVA	(New Collector Sewers)	16.979	17.790
IVB	(New Interceptor Sewers)	17.923	18.526
V	(Combined Sewer Overflows)	18.262	19.357
	Total I-V	95.902	100.422
VI	(Control of Stormwater)	54.133	57.250
	Total I-VI	150.035	157.672

Attachments 2 and 3 provide listings of the relative needs reported by EPA and the States.

To a large extent annual state construction grant allocations are established by Congress on the basis of a formula derived from each State's proportionate share of the above categories. As an example, in 1977 each State's allocation was determined on the basis of (a) one-fourth on the full needs of a State in all categories, (b) one-half on the partial needs for a State in Categories I, II, and IVB, and (c) one-fourth on the estimated 1975 census population for each State.

In addition to establishing a base for the distribution of Federal funds among the States, the needs survey developed by each individual state becomes the basis for determining priority for funding municipal water pollution control projects.

ANNUAL STATE STRATEGY AND PROJECT PRIORITY LIST

As part of the State Continuous Planning Process, each State submits to the Regional Office an annual strategy for water pollution control. An important element of the annual strategy is a listing of the 5-year funding priorities for municipal treatment works construction. The 5-year list generally reflects the inventory of municipal needs which the State also maintains under the planning process. The needs survey discussed above provides the basis for the inventory.

These priorities are determined in accordance with a Priority System developed by the State and approved by the EPA Regional Office. This system serves as the basic management tool for the Federal, State, and local governments to plan, budget, and oversee the construction grants program. It describes a methodology for establishing project ratings by applying priority criteria based on (a) severity of the pollution problem, (b) existing population affected, and (c) preservation of high quality waters. The relative influence of these criteria for assigning project priority is determined by the State. However, it must be designed to provide funding for the most serious existing municipal water pollution needs that are necessary to meet the water quality objectives of the Act. The priority system also sets forth the administrative and public participation procedures for developing and revising the State priority list.

Using the municipal needs inventory as a basis, the 5-year project priority list is composed of two parts, a fundable portion and a planning portion. The fundable portion includes only those projects for which a grant award is planned during the first year and cannot exceed the total appropriated allocations available during that year. The planning portion of the list includes all projects outside the fundable portion that may receive funding during the 5-year period from anticipated allotment levels. Attachment 4 provides an example of the information contained on a proposed priority list for Fiscal Year 1979 for the Commonwealth of Virginia.

Municipalities and their consultants are integral to the development of the list by providing the State with up-to-date cost estimates and amounts of grants to be requested, proposed schedules for funding, and other information used in determining priority. Municipalities also play an important role in the public participation process. Prior to submission of the list to EPA Regional Office, the State conducts a public hearing on the proposed list in which it elicits comments from affected municipalities, environmental groups, consultants, and the general public. All significant issues resulting from the hearing must be addressed before the State submits the final priority list to EPA.

The priority system and list are reviewed by the EPA Regional Office for conformity with Agency priority system regulations, adequacy of public participation, project eligibility, and compliance with the enforceable provisions of the Act. If any project is determined to be ineligible for Federal participation, the State is required to remove the project from the list. EPA may also propose the removal of a project when there is reason to believe that it will not result in compliance with the enforceable requirement of the Act.

STEP 1 PROCESS (FACILITY PLANNING)

After the EPA Regional Office concurs in the priority list, municipalities in the fundable portion of the list are requested by the State to submit a Step 1 grant application and a plan of study. To assist in the preparation of the Step 1 application and plan of study, the State conducts a preapplication conference outlining the responsibilities of the municipality, State, and EPA relative to the project and grants program. EPA Regional staff join the State in conducting preapplication conferences for the most complex and environmentally sensitive projects.

The plan of study sets forth the scope of work to be undertaken in preparing a facility plan and contains the following items:

(a) Delineation of boundaries for the planning area indicating the water quality problem to be addressed.

(b) Identification of the municipal jurisdiction(s) that will conduct the study.

(c) The nature and scope of the Step 1 project, including a schedule for completion of specific tasks and any proposed subagreements for personal and professional services such as consulting engineers.

(d) An itemized estimate and description of project costs.

The Step 1 application and plan of study are submitted to the State Agency for review. It is important to note that the application must be accompanied by comments from appropriate State and Regional clearinghouses. These clearinghouses are established in order to provide early contact between the applicants and other governmental agencies and to coordinate the proposed project with related projects of other agencies. The purpose of the clearinghouse review is to ensure that other governmental interests, not directly related to the water pollution control effort, have an opportunity to review the proposed grant as it impacts their area of specific responsibility. For instance a proposed water pollution control project may impact a proposed transportation control program. By exposing potential conflicts at the clearinghouse stage, the issues can be addressed and resolved early in the process. In order to approve and forward the application and plan of study to EPA, the State must ensure consistency with plans developed under the continuous planning process, compliance with State and local regulatory requirements, and conformity with national program requirements.

The EPA review of the application and plan of study is similar to the State review but also includes a review for consistency with the priority list. Prior to the award of the grant, EPA may conduct a preaward audit to determine the fiscal management capabilities of the municipality and the conformity of its professional services procurement procedures with EPA regulations.

If the application meets all applicable requirements, a grant is awarded to the municipality by the EPA Regional Administrator or his designee. The grant award document is termed a "grant agreement" and constitutes a legally binding contract between the Federal government and the grantee once executed. The grant agreement defines the amount of the grant, scope of the project, schedule for completion, and any other grant terms and conditions. The Regional Office notifies the clearinghouses and the Washington Headquarters of the award. In turn Headquarters notifies Senators and the Congressmen in whose district the award is made.

After the grant agreement is executed by the authorized representative, the actual work of preparing the facility plan may be initiated by the grantee's consultant. The facility plan is the basic document which defines the extent of the pollution problem, investigates alternatives for solving the problem, describes environmental impacts associated with each alternative, develops cost-effective analysis of proposed alternatives, and recommends one of the alternatives to be implemented for solving the problem.

Probably the most difficult aspect of developing a facility plan is defining the most cost-effective alternative which is environmentally acceptable. Traditionally, the engineering profession has favored centralized systems that collect the wastewater, convey it to a chemical or biological sewage treatment plant, provide treatment, and discharge the effluent to a receiving water body. This conventional approach may be the most cost-effective solution. However, EPA is requiring a more detailed investigation of less conventional alternatives with an objective of reducing cost and providing a more environmentally acceptable means of disposing of wastewater. The principal purpose of the alternative investigation is the detailed consideration of less conventional alternatives such as land application of wastewater.

Although land application has wide acceptance in other parts of the world, a psychological and emotional aversion to recycling human wastes has created an overwhelming opposition to land application in the United States. Overcoming this opposition by providing accurate scientific and health related data and information becomes the greatest challenge of the facility plan. Where land application is a viable alternative, all this information must be provided in the facility plan.

Public participation is a requirement throughout the grants process. It is particularly sensitive during the facility planning stage since this is where the process is most adaptable to incorporating meaningful suggestions. Ultimately, the alternative selected for implementation must be operated and maintained by the public it is designed to serve. Therefore, it is imperative that the public be given the open and free opportunity to make their viewpoints known during the planning phase of the project.

Local governments must provide 25 percent of the capital cost of construction and 100 percent of the operation and maintenance cost of the facility. This cost is usually provided for by assessing user charges to the customers in the service area. An important feature of the facility plan is to provide accurate estimates of the project cost so that the users of the system will know in advance what charges they will be assessed when the system is operational.

The facility plan must also identify the local governmental and institutional responsibilities for implementing the recommended alternative. The political leaders in the planning area must understand the plan and be aware of their responsibilities in implementing the plan.

The National Environmental Policy Act of 1969 (NEPA) requires all Federal agencies to prepare environmental impact statements (EIS) for projects which have significant adverse environmental effects or environmental impacts which are likely to be highly controversial. Therefore, to allow EPA to decide if an EIS is to be prepared, grantees are required to prepare an environmental assessment (EA), as a part of the facility plan, which includes an environmental inventory, an evaluation of the alternatives, and proposed mitigative measures. The EA is similar in content to an EIS and is required for all projects.

If, after reviewing the completed facility plan (which contains the environmental assessment), EPA decides that the proposed project does not warrant the preparation of an environmental impact statement, EPA will issue a "negative declaration" and appropriate public notices. On the other hand, if EPA decides that an EIS is warranted, a "notice of intent" will be issued and made public. EPA will then prepare a draft EIS extracting as much information as possible from the EA.

A variation of this procedure is used for those projects which are known in advance to have significant or controversial environmental impacts. In such cases, and with all parties agreeing, a facility plan and an environmental impact statement will be prepared simultaneously. This procedure, known as "piggybacking", is intended to save time and money yet satisfy all the requirements of the construction grants program.

The Water Pollution Control Act of 1972 established the National Pollution Discharge Elimination System (NPDES) as the enforcement mechanism for achieving water quality standards. The NPDES permits issued under the system are applicable to all municipal discharges and the municipalities must comply with their provisions. Where basin, areawide waste management or facility plans have been established, the permits will require compliance with such plans.

The NPDES permit may contain limitations, conditions or schedules which will require the municipality to undertake the construction of a wastewater treatment facility. All alternatives in a facility plan must meet the conditions of an NPDES permit.

STEP 2 PROCESS (DESIGN)

After approval of the facility plan, a Step 2 design application may be prepared by the municipality. The application must be accompanied by clearinghouse comments and assurances for complying with Federal regulations governing user charges, industrial cost recovery, and the availability of facility site. If all requirements are met, a grant is awarded through the usual grant agreement and notification procedures.

User charges are those fees assessed to all system users, both residential and industrial, to defray the costs of operation and maintenance. Industrial cost recovery provisions require industrial users to repay their proportionate share of the Federal grant. Half of the recovered funds are placed by the grantee in a special fund to be used for future expansion and reconstruction of the facility. The other half of the funds are periodically returned to EPA and subsequently to the U.S. Treasury.

The end product of the Step 2 detailed design is a set of plans (drawings), specifications, and detailed construction cost estimates which are suitable for bidding and construction purposes. Since large sums of public funds will be spent on constructing the project, it is of the utmost importance that the plans, specifications, and estimates be as complete and accurate as possible. Plans, specifications and estimates must be submitted to obtain the final Step 2 grant payment, and approved plans and specifications must be submitted to obtain a Step 3 grant.

The State and EPA review the completed Step 2 work for compatibility with technical requirements to ensure that the proposed facility, if constructed in accordance with the plans and specifications, will achieve effluent limitation or water quality standards required by the NPDES permit. The review is also to ensure that sound engineering and design principles were employed by the municipality and consulting engineer and that the plans and specifications are suitable for bidding purposes. Upon approval of the plans and specifications the grantee is notified of any special conditions resulting from the review that may be imposed by EPA.

STEP 3 PROCESS (CONSTRUCTION)

After receiving notification that the project plans and specifications have been approved, the municipality may prepare a Step 3 construction application to be submitted to the State and EPA. The applicant must submit any proposed professional services contracts or plans for utilizing its own engineering and inspection forces. Assurances must also be submitted for participating in the national flood insurance program, completing sewer use ordinances, finalizing operation and maintenance plans, and obtaining the non-Federal share.

The State and EPA review the application and supporting documents for conformity with applicable regulations. When all requirements have been met, a grant is awarded with the following special conditions related to specific levels of payments to the grantee:

(a) User charges and Industrial Cost Recovery System - Not more than 50 percent of the grant may be paid until the grantee submits evidence of timely development of UC & ICR System and not more than 80 percent until the systems are approved.

(b) Operation and Maintenance Manual - Not more than 50 percent of grant may be paid until the draft manual has been submitted and not more than 90 percent until a satisfactory manual has been furnished.

(c) Sewer Use Ordinance - Not more than 80 percent of the grant may be paid until the sewer use ordinance is approved.

After executing the grant agreement, the grantee is authorized to advertise and receive bids. The grantee's advertising, bidding, and selection process must be in strict accordance with Federal competitive bidding and procurement procedures. The contractor selected by the grantee must be the lowest responsible bidder.

Contracts cannot be awarded without the approval of EPA. Bid documents are forwarded through the State agency to EPA for approval and adjustment in the grant to reflect the bid amounts and a reasonable contingency to cover any changes in the project. No grant may be increased without the expressed approval of the State. Any decreases in the grant are credited to the State allotment.

The proposed award of construction contracts may result in a protest from an unsuccessful bidder, equipment suppliers, or others with vested interests in the selection process. It is the responsibility of the grantee to resolve the protest, but the State and EPA must be advised of the method of resolution. If EPA determines that the selection process was in accordance with competitive bidding requirements, the grantee will be advised to award contracts. Normally a preconstruction conference is held between the grantee, contractor, State Agency, and EPA to discuss the responsibilities of each party.

During the course of construction, contracts may be modified to correct errors in the plans and specifications or changed conditions. Any change that is minor or emergency in nature is usually initiated without prior State and EPA approval. However, changes which significantly alter the size, location, and scope of the project or require additional Federal funds must receive prior approval. Approved change orders resulting in construction cost increases are generally paid out of the contingency fund authorized in the grant.

Management of the contracts and quality of construction is a responsibility that must be ensured by the grantee through a rigorous supervision and inspection program. To ensure that the grantee is performing its management responsibilities, the State or EPA (or EPA's agent, the U.S. Army Corps of Engineers) conduct periodic inspections during the course of construction. Deficiencies found during these inspections must be corrected or grant payments may be withheld.

Throughout the construction phase, the grantee may request reimburement of costs incurred at predetermined frequencies outlined in the grant agreement. Requirements for UC/ICR systems, O&M manual, and sewer use ordinances are checked for compliance with payment restriction policies. Generally, EPA withholds 5 to 10 percent of payments until a final inspection is conducted by the Agency's project manager to determine that the project has been constructed in accordance with the plans and specifications and that all grant conditions have been met. The project manager prepares a final inspection report stating the finding of the inspection and recommending either final payment or withholding of payment until deficiencies are corrected.

After final payment, the project is referred to the EPA Audit Office to ensure that all costs are eligible for Federal participation. Grantees are afforded an opportunity to comment on the audit findings prior to completion of the final audit report. The final audit report is then referred to the responsible EPA Regional Office for resolution. Any monies owed the Federal Government as the result of an audit exception must be returned to EPA. Final adjustments in the grant are made through the usual grant amendment procedures.

PROGRAM ISSUES

The Construction Grants Program is the largest public assistance program in the United States. The Federal grant to fund 75 percent of the capital costs of needed pollution control works is enabling local municipalities to meet the goals of the Water Pollution Control Act at an accelerated pace. However, a program that involves $25.5 billion and a short timeframe (5 years) for implementation creates special management problems. The following is a discussion of some of the problems and issues facing the construction grants program.

Program Complexity

The laws and regulations governing the construction grants program are numerous and complex. All levels of government are involved in the program. Time and resources are often misdirected because of lack of understanding of the program requirements. It is a constant challenge to program managers to ensure that each level of government are knowledgeable with respect to the program so that they can exercise this responsibility.

Growth

The first priority of the construction grants program is to fund projects that solve an existing pollution problem. Since wastewater control works are a necessary prerequisite for land development, the program is under constant pressure to fund projects which are oriented toward growth rather than pollution abatement. Reasonable amounts of growth are eligible to be funded through the program. It is the responsibility of the program management to ensure that the growth built into a project is cost-effective and environmentally sound and not due to land speculation and development pressures.

Program Resources

There are approximately 1100 manyears in EPA devoted to managing the Construction Grants Program. This is not adequate to effectively manage the program and ensure all requirements are met. EPA is now engaged in an accelerated effort to delegate certain responsibilities to state governments. Financial resources to hire employees at the state level are being made available through the construction grants allocations. State delegations will make more resources available to the program and also eliminate duplicative reviews at the State and Federal level. Also, through an Inter-Agency Agreement, the U.S. Army Corps of Engineers are assuming an active role in the inspection of construction works. This review is geared to ensuring that projects are built according to design and specifications.

High Project Costs

Project costs are accelerating rapidly. Although the allocations to the program are considerable, the needs still exceed the available funds. Continuing efforts are underway to ensure the least costly environmentally sound alternatives are developed and proposed for implementation. The 1977 Amendments encourage the use of innovative and alternative technologies by providing financial incentives. These incentives are designed to promote the recycling of waste products not only to conserve valuable resources but also to reduce project costs.

Pretreatment

Wastewater treatment systems in the United States often receive industrial wastes as well as domestic wastes. Some systems administrated by municipal government have failed to achieve their design expectations because the industrial waste was not compatible with the municipal sewage treatment plant. Also, because of this incompatibility, the resulting sludges are often contaminated with heavy metals and toxics making the recycling of sewage sludge restrictive. EPA has recently issued pretreatment regulations (designed to alleviate the problem), but because of the magnitude of the problem, it will be several years before the full benefit of a pretreatment program will be realized. The cost of industrial pretreatment is not eligible for a grant and must be borne entirely by the industry.

Public Participation

Public participation is a necessary ingredient at all phases of the program. The public must eventually pay for any project that is implemented and must also live with the environmental and social consequences of the project. Often the public does not become concerned about a project until it is almost ready to go under construction. At this stage numerous commitments have been made and it is difficult to alter the makeup of the project. The goal of EPA is to encourage public particiation early in the planning phase where issues are most susceptible to negotiation. Identifying, educating, and maintaining communication with affected citizens and public interest groups continues to be a programmatic goal of the grants program.

REFERENCES

1. Code of Federal Regulations, Title 40, Subpart B, Parts 30 and 35, July 1, 1977.

2. "Cost Estimates for Construction of Publicly-Owned Wastewater Treatment Facilities, 1976 Needs Survey", U.S. Environmental Protection Agency, February, 1977.

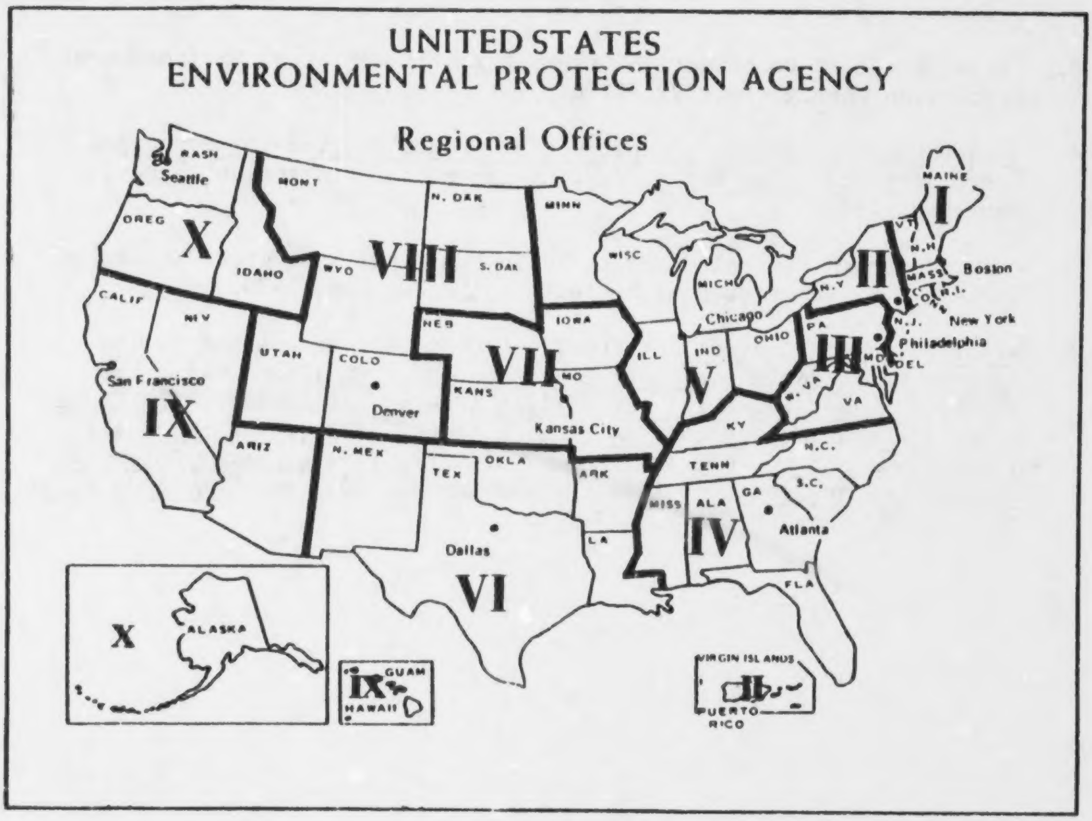
3. "Final Draft of Construction Grants Regulations", U.S. Environmental Protection Agency, July 29, 1978.

4. Handbook of Procedures - Construction Grants Program for Municipal Wastewater Treatment Works, U.S. Environmental Protection Agency, February, 1976.

5. "How to Obtain Federal Grant to Build Municipal Wastewater Treatment Works", U.S. Environmental Protection Agency, May, 1976.

6. Water Pollution Control Act (Public Law 84-660) as amended by The Federal Water Pollution Control Act Amendment of 1961 (Public Law 87-88), The Water Quality Act of 1965 (Public Law 89-234), The Clean Water Restoration Act of 1966 (Public Law 89-753), The Water Quality Improvement Act of 1970 (Public Law 91-224), Federal Water Pollution Control Act Amendment of 1972 (Public Law 92-500), and The Clean Water Act of 1977 (Public Law 95-217).

ATTACHMENT 1


UNITED STATES
ENVIRONMENTAL PROTECTION AGENCY

Regional Offices

ATTACHMENT 2

FEBRUARY 10, 1977
TABLE I

1976 NEEDS SURVEY

EPA ASSESSMENTS FOR CONSTRUCTION OF PUBLICLY-OWNED WASTEWATER TREATMENT FACILITIES
MILLIONS OF 1976 DOLLARS

CATEGORIES I THRU V

	TOTAL	I	II	III A	III B	IV A	IV B	V
ALABAMA	865	164	254	65	2	206	172	0
ALASKA	327	110	25	1	0	68	118	3
ARIZONA	458	216	5	1	0	103	131	0
ARKANSAS	517	78	207	21	0	107	56	46
CALIFORNIA	5,544	2,187	1,184	90	9	838	847	385
COLORADO	697	90	300	20	237	8	37	3
CONNECTICUT	1,273	129	63	31	10	467	183	387
DELAWARE	269	0	94	3	1	32	96	39
DIST. OF COLUM.	227	0	12	58	0	0	7	149
FLORIDA	3,726	286	1,209	95	6	1,139	989	0
GEORGIA	1,635	98	607	41	0	211	374	301
HAWAII	752	320	0	0	0	128	303	0
IDAHO	437	131	47	16	0	94	138	8
ILLINOIS	5,365	51	1,775	103	58	333	459	2,585
INDIANA	3,164	80	753	201	57	458	380	1,232
IOWA	1,035	236	410	56	0	92	148	90
KANSAS	679	271	25	33	0	77	162	107
KENTUCKY	1,283	46	427	64	0	226	385	132
LOUISIANA	707	314	75	109	2	97	107	0
MAINE	962	119	16	14	3	183	142	482
MARYLAND	2,912	7	1,201	21	1,128	60	444	47
MASSACHUSETTS	3,400	705	287	16	46	834	633	876
MICHIGAN	5,133	29	901	73	578	1,230	973	1,347
MINNESOTA	1,610	0	874	24	10	170	307	223
MISSISSIPPI	750	65	370	109	2	52	149	0
MISSOURI	2,590	659	7	19	0	159	452	1,292
MONTANA	126	43	15	6	1	17	42	0
NEBRASKA	409	163	24	4	0	14	57	146
NEVADA	265	51	118	2	0	26	51	14
NEW HAMPSHIRE	1,041	169	78	6	3	216	258	307
NEW JERSEY	3,252	803	714	125	27	397	579	604
NEW MEXICO	148	66	32	4	0	23	20	0
NEW YORK	14,514	2,531	1,241	148	2,525	3,248	2,155	2,663
NORTH CAROLINA	1,213	72	522	57	0	147	412	0
NORTH DAKOTA	73	39	0	0	1	5	13	12
OHIO	7,192	28	2,062	629	357	974	1,373	1,765
OKLAHOMA	516	8	279	32	12	66	116	0
OREGON	1,382	93	314	116	34	243	370	210
PENNSYLVANIA	3,806	258	1,257	33	0	951	485	820
RHODE ISLAND	722	57	78	8	0	230	82	264
SOUTH CAROLINA	957	180	226	38	0	168	342	0
SOUTH DAKOTA	137	23	86	4	0	3	18	1
TENNESSEE	1,279	89	393	52	0	307	252	183
TEXAS	2,814	8	1,375	178	61	460	680	48
UTAH	270	0	167	42	0	35	25	0
VERMONT	388	45	67	1	5	78	27	161
VIRGINIA	1,583	403	307	29	1	194	402	244
WASHINGTON	1,999	406	239	107	136	392	374	341
WEST VIRGINIA	2,184	260	86	7	0	626	776	426
WISCONSIN	1,976	165	414	72	143	421	481	277
WYOMING	67	60	0	1	0	0	4	0
AMERICAN SAMOA	59	28	0	0	2	7	22	0
GUAM	94	23	0	0	0	31	17	22
PUERTO RICO	898	352	19	6	11	240	268	0
PAC. TR. TERR.	158	110	8	0	0	35	3	0
VIRGIN ISLANDS	45	4	13	0	0	23	3	0
U.S. TOTALS	95,902	12,955	21,279	3,017	5,486	16,979	17,923	18,261

NOTE: SUM OF ENTRIES MAY NOT EQUAL TOTALS DUE TO ROUNDING

ATTACHMENT 3

FEBRUARY 10, 1977
TABLE 2

1976 NEEDS SURVEY

STATE ESTIMATES FOR CONSTRUCTION OF PUBLICLY-OWNED WASTEWATER TREATMENT FACILITIES
MILLIONS OF 1976 DOLLARS

CATEGORIES I THRU V

	TOTAL	I	II	III A	III B	IV A	IV B	V
ALABAMA	865	164	254	65	2	206	172	0
ALASKA	329	110	25	1	0	68	118	5
ARIZONA	458	216	5	1	0	103	131	0
ARKANSAS	519	78	208	21	0	107	56	46
CALIFORNIA	6,420	2,187	1,184	90	9	838	847	1,261
COLORADO	707	91	300	29	237	8	37	3
CONNECTICUT	1,293	145	63	31	10	470	185	387
DELAWARE	269	0	94	3	1	32	96	39
DIST. OF COLUM.	227	0	12	58	0	0	7	149
FLORIDA	3,726	286	1,209	95	6	1,139	989	0
GEORGIA	1,635	98	607	41	0	211	374	301
HAWAII	763	331	0	0	0	128	302	0
IDAHO	440	133	47	16	0	96	138	0
ILLINOIS	5,379	51	1,776	103	80	344	459	2,505
INDIANA	3,245	85	769	207	85	482	402	1,232
IOWA	1,085	236	410	56	0	92	198	90
KANSAS	838	299	125	37	1	78	187	107
KENTUCKY	1,283	46	427	64	0	226	385	132
LOUISIANA	707	314	75	109	2	97	107	0
MAINE	962	119	16	14	3	183	142	482
MARYLAND	2,912	7	1,201	21	1,128	60	444	47
MASSACHUSETTS	3,400	705	287	16	46	834	633	876
MICHIGAN	5,146	29	905	77	582	1,230	973	1,347
MINNESOTA	1,610	0	874	24	10	170	307	223
MISSISSIPPI	750	65	370	109	2	52	149	0
MISSOURI	2,590	659	7	19	0	159	452	1,292
MONTANA	127	43	15	6	1	17	42	0
NEBRASKA	410	163	24	4	0	14	57	146
NEVADA	265	51	118	2	0	26	51	14
NEW HAMPSHIRE	1,041	169	78	6	3	216	258	307
NEW JERSEY	3,271	796	712	140	27	399	589	604
NEW MEXICO	148	66	32	4	0	23	28	0
NEW YORK	16,510	2,679	1,870	148	2,648	3,973	2,525	2,663
NORTH CAROLINA	1,213	72	522	57	0	147	412	0
NORTH DAKOTA	77	42	0	0	1	5	14	12
OHIO	7,270	28	2,062	629	432	975	1,375	1,765
OKLAHOMA	524	9	279	33	12	72	116	0
OREGON	1,486	93	314	219	34	243	370	210
PENNSYLVANIA	3,806	258	1,257	33	0	951	485	820
RHODE ISLAND	772	57	88	13	0	230	125	264
SOUTH CAROLINA	957	180	226	38	0	168	342	0
SOUTH DAKOTA	137	23	86	4	0	3	18	1
TENNESSEE	1,279	89	393	52	0	307	252	183
TEXAS	2,881	8	1,392	228	61	460	680	48
UTAH	270	0	167	42	0	35	25	0
VERMONT	392	46	67	4	6	78	27	161
VIRGINIA	1,757	403	307	156	21	211	412	244
WASHINGTON	2,437	406	239	521	136	392	399	341
WEST VIRGINIA	2,184	260	86	7	0	626	776	426
WISCONSIN	2,194	165	414	72	143	421	481	495
WYOMING	67	60	0	1	0	0	4	0
AMERICAN SAMOA	59	28	0	0	2	7	22	0
GUAM	94	23	0	0	0	31	17	22
PUERTO RICO	1,009	384	20	13	24	250	308	0
PAC. TR. TERR.	150	110	0	0	0	35	3	0
VIRGIN ISLANDS	52	9	13	0	0	23	5	0
U.S. TOTALS	100,422	13,199	22,849	3,765	5,732	17,790	18,526	19,357

NOTE: SUM OF ENTRIES MAY NOT EQUAL TOTALS DUE TO ROUNDING

ATTACHMENT 4

AUGUST 1978

Draft

VIRGINIA'S CONSTRUCTION GRANTS PRIORITY LIST

FISCAL YEAR 79 & EXTENDED PRIORITY FUNDING

COMMONWEALTH OF VIRGINIA

RANKING POINTS	APPLICANTS NAME ADDRESS	PARENT PROJECT NUMBER PROJECT NUMBER	PROJECT DESCRIPTION (NPDES NUMBER)	SMALL COMMUNITY	INNOVATIVE COST ALTERNATIVE COST	NEEDS NUMBER ELIGIBLE COST BY CATEGORY	FY - 79		EXTENDED PRIORITY FUNDING LIST							
							FY - 79 STEP	FY - 79 GRANT FUNDS	FY - 80 STEP	FY - 80 GRANT FUNDS	FY - 81 STEP	FY - 81 GRANT FUNDS	FY - 82 STEP	FY - 82 GRANT FUNDS	FY - 83 STEP	FY - 83 GRANT FUNDS
91 333.33	Appomattox, Town of Municipal Building Appomattox, VA 24522	536	STP Expansion & Upgrade (VA0020249)	X		0142-003 II 1.200 IIIA .009 IVA .370 IVB .445					C	$1.443m				
92 284.09	HRSD/Atlantic P. O. Box 5000 Virginia Beach, VA 23455	545 545-04	New STP (VA0062618)			0308-012 I 83.808 IVB 19.529	III	$9.4m	III	$15.1m	III	$24.762m				
93 279.05	HRSD/Newtown Road P. O. Box 5000 Virginia Beach, VA 23455	560	Interceptor					PRO		JECT F		ULLY F		UNDED		
94 277.86	Strasburg, Town of Town Hall Strasburg, VA 22657	527	STP Expansion & Upgrade (VA0020311)	X		0064-001 I .741 IIIA .386 IVA .048 IVB .052					III	$1.816m				
95 277.85	Mount Jackson, Town of P. O. Box 382 Mount Jackson, VA 22842	538	STP Expansion & Upgrade	X		0061-001 I .530 IVA .032 IVB .142					C	$0.368m				
96 277.84	Clinchport, Town of Municipal Building Clinchport, VA 24227	539	STP Expansion & Upgrade					PRO		JECT F		ULLY F		UNDED		
97 277.80	Weber City, Town of Municipal Building Weber City, VA 24251	542	New System								II	$0.329m			III	$4.27m
98 277.79	Independence, Town of P. O. Box 99 Independence, VA 24348	543	STP Expansion & Upgrade (VA0031691)	X		0129-002 I .204 IIIA .060					C	$1.398m				
99 277.78	Orange, Town of Municipal Building Orange, VA 22960	544	STP Expansion & Upgrade (VA0021385)	X		0100-001 I 1.430 IIIA .011 IVA .213 IVB .266					II	$0.02m	III	$1.6m		
100 277.78	Middleton, Town of P. O. Box 175 Middleton, VA 22645	547	STP Expansion & Upgrade	X							C	$0.361m				

WATER QUALITY PROGRAMS OF THE VIRGINIA STATE WATER CONTROL BOARD

R. V. Davis
Executive Secretary
Virginia State Water Control Board

ABSTRACT

The Virginia State Water Control Board was created in 1946 to protect the high quality of the State's waters, to prevent any increase in pollution and to reduce existing pollution. The Agency has grown dramatically in size and scope since then and has a broad legislative mandate in the areas of water conservation and groundwater management as well as water quality management. Virginia has been blessed with great wealth in her water resources and State laws and programs have kept pace with the times to ensure that abundant, high quality water is available for future generations to utilize and to enjoy.

Description of Virginia

Virginia, in terms of area, population and industry, is close to being a median state. The Commonwealth has a land area of 40,815 square miles and a population of 5,135,000 (2.4 percent of the U.S. total). Population and economic growth in the Southeast states, and particularly Virginia, have since the 1960's consistently outpaced the nation. Since 1970, the State's population increased 10.4% compared to the national average of 6.4%. Similarly, Virginia's 1976 per capita income was $6,341, the highest in the Southern States and almost identical to the national average of $6,399.

The geography of the State ranges from the Tidewater, only a few feet above sea level, through a 100-mile Piedmont band into the Blue Ridge and Shenandoah Mountains. The land area of the State is divided almost evenly among these three geographic regions. The State has a uniformly distributed rainfall of approximately 43 inches annually, but has ranged from a low of 25 inches to a high of 53 inches a year. The State has nine major river basins (most of which are interstate in nature involving North Carolina, Tennessee, Kentucky, West Virginia, Maryland and the District of Columbia), 120 miles of coastline along the Atlantic; 1,900,000 acres of saltwater in the Chesapeake Bay; and the two natural lakes (Lake Drummond and Mountain Lake), six major impoundments and numerous streams and small ponds account for 429,000 acres of freshwater.

Industry in Virginia is diverse, constantly expanding and is widely dispersed throughout the State. Virginia has extensive agriculture, with emphasis on orchards, peanuts, tobacco, small grain and meat production. A substantial portion of the State is covered by forests which supply raw materials for the State's numerous paper, hardboard mills and sawmills.

History and Organization of the Virginia State Water Control Board

The Virginia General Assembly has given the State Water Control Board a broad legislative charter in the general areas of water quality management, conservation of water resources and management of groundwater. The earliest of the three statutes -- the State Water Control Law -- sets forth the policy of the Commonwealth to protect the high quality of the State's waters, to prevent any increase in pollution, and to reduce existing pollution in order to provide for the health, safety and welfare of its citizens. The 1946 law contained all the ingredients for the operation of a comprehensive program to accomplish these objectives and while the Law has subsequently been amended a number of times, its basic approach has remained unchanged. The newly created State Water Control Board was to accomplish the objectives of cleaning up existing pollution and preventing new pollution by:

1] setting standards of water quality that would permit all reasonable and necessary use of streams; and

2] requiring municipalities, industries and other waste dischargers to reduce pollutants, such that the established standards would be met.

The 1966 General Assembly enacted the Water Resources Planning Act which directed that the Division of Water Resources of the Department of Conservation and Economic Development "shall formulate a coordinated policy for the use and control of all the water resources of the State," prepare "comprehensive water and related land resource plans for each of the major river basins," and recommend to the General Assembly additional legislation which may be deemed necessary or desirable for the accomplishment of such plans or programs.

The State Water Control Board has grown dramatically, both in size and scope since 1946 with one of the major organizational changes coming in July 1972 when the General Assembly merged the Division of Water Resources with the State Water Control Board. As a result of this merger, the Water Control Board was given authority for both water quality and planning the development, conservation and utilization of the State's water resources. In 1973, the Board's responsibility was further broadened by the passage of the Groundwater Act of 1973 which authorizes the Agency to allocate groundwater resources among competing users in those areas of the State in which demand does or may exceed supply.

Currently, the agency expends about 75% of its efforts in the area of water pollution control and abatement; 20% in the development of water resource and water supply plans; 4% in groundwater management programs; and 1% in flood damage reduction and flood insurance. (View graph 1)

Structurally, the agency is responsible to the Secretary of Commerce and Resources and a seven-member citizen board appointed by the Governor. (View Graph 2) In late 1972, the staff of the Water Control Board underwent a sweeping reorganization which ultimately resulted in the agency's current structure of a headquarters operation in Richmond and six regional offices located strategically throughout the State. (View Graph 3) Four technical headquarters bureaus -- Applied Technology (industrial, municipal and agricultural permits; training); Enforcement (legal matters involved in permit compliance); Water Control Management (resource planning); and Surveillance and Field Studies (pollution complaint investigation, ecological studies, and monitoring) -- whose primary functions are to set policy, provide procedural guidance and see that programs are carried out uniformly throughout the State, are located organizationally under a Deputy Executive Secretary. Also under the Deputy Executive Secretary are the six regional offices which are responsible for program implementation. The regional offices were established to better serve the citizens of the Commonwealth by providing more accessibility at the local level and have gone a long way toward assuring that the State Water Control Board is responsive to regional problems and can offer technical assistance to localities. Approximately one half of the Board's 350 authorized positions (plus eight supported by federal funds) are located in the regional offices. (View Graph 4)

Located under an Assistant Executive Secretary are the Bureau of Program Management and Administration (program planning, automated data processing, personnel); Division of Construction Grants and the Public Information Office.

The Agency's budget for fiscal year 1978 totalled approximately $7.1 million -- $5.25 million from State appropriations and $1.8 million from federal program grants. (View Graph 5) I think it is interesting to note that, with a State population of 5,135,000 the total cost for operating the Board's programs was only $1.38 per capita; and if you consider only State funds, the cost is only $1.02 for every man, woman and child in the State per year!

From an operational standpoint, about 71% of the budget goes to pay the 350-plus individuals employed by the agency, with the remaining 29% utilized for property rental, travel, supplies, equipment, etc. (View Graph 6) From a programmatic standpoint, the budget is divided as follows: 24% for administration of the NPDES program; 14% for monitoring and surveillance; 13% for operations and maintenance (which includes property management, insurance, personnel functions and staff training); 10% for water resource and water quality planning; 21% for services (including automated data processing, accounting, drafting and reproduction, procurement and supply, and the agency library); 4% for groundwater management programs; 5% for non-point source programs; 6% for construction grant programs; and 3% for flood protection programs. (View Graph 7)

I have attempted to outline for you basically how the agency is organized and the State laws under which it operates. At this point, I would like to discuss some of the major programs of the State Water Control Board with the emphasis on water quality.

Program Planning

The program planning process is an integral part of the State Water Control Board's operation. The State Water Strategy is the initial guidance policy for development of individual work plans. The Strategy is developed from: (View Graph 8)

1] Requirements of P.L. 92-500

2] Requirements of existing State statutes

3] Requirements of newly passed legislation - State and Federal

4] Identification, by the agency through regional profiles and problem analysis, of areas which will significantly impact the Commonwealth on water quality and resource issues.

From this strategy, the Agency develops program priorities and the internal work plans. The work plans reflect detailed project information to include project narrative descriptions, milestones, work tasks, management responsibility, manpower resources required and program interfaces. The final work plan guides the operations of the Board through the fiscal year towards the accomplishment of the goals and objectives established through the identification of long-range planning issues.

The work plan is updated annually to guide the Agency in maximum use of its financial, personnel and other resources to achieve the most effective results in implementing and enforcing the State and Federal laws under which the Board operates.

The Municipal Construction Grants Program

Much of the improvement in water quality in recent years can be attributed to the construction of new and more advanced waste treatment facilities. The Water Control Board, working closely with the U.S. Environmental Protection Agency, is responsible for administering the federal grants program for the construction of municipal sewerage facilities in Virginia. Since 1956, when the program began, Virginia has received $605 million in Federal funds to assist communities in constructing needed sewerage treatment facilities by providing grants of 75% of the total eligible cost. (Until 1975, the Commonwealth provided supplemental grants ranging from 5% to 20%; however, the General Assembly did not authorize an appropriation for continuation of the program.) The grants program has been an essential element of the Board's overall pollution abatement program and as of September 7, 1978, 46 projects (totalling more than $359 million in construction costs) were under construction in Virginia, with another 6 projects (over $20 million in construction costs) in the bidding stage.

The Clean Water Act of 1977 authorizes construction grant funding in the amount of $4.5 billion for each of fiscal years 1978 through 1982. The Appropriations Committee authorized the release of funding on February 22, 1978. Based on the allotment formula, Virginia will receive $88,209,000 or 1.9602% of the total in each of the fiscal years. Based on the allocation formula, the amount of funds that Virginia will receive in the next five years should be approximately $440 million. (View Graph 9) The Board foresees a need of some $802 million in grant funds (which equals almost $1 billion in construction) to construct needed sewerage works during that period. (View Graph 10) This shortfall, coupled with a number of major, multi-million dollar projects which require funding, will make the Board's task of equitable fund distribution extremely difficult. (View Graph 11)

Since clean water is mandated by federal and state laws, we have no choice but to implement the construction of the appropriate facilities as expeditiously as possible. It is obvious that the State needs some form of alternative financing and cost improved efficiency in plant design.

Toward that end, the Chairman of the State Water Control Board called upon the Virginia Society of Professional Engineers to organize an effort exploring the possibility of reducing the cost of waste treatment facilities through innovative cost control methods, plus any alternatives that they might find applicable to this problem. In addition, the Chairman requested the Governor to consider empaneling an advisory committee to investigate and implement the best possible system of local funding for rural and municipal waste treatment facilities to augment the insufficient federal resources. Both the Governor and the Professional Engineers have reacted favorably to the requests and committees have been formed to study the problem.

Pollution Response Program

The most well-known of the Board's programs is probably that of oil spill, fish kill and pollution complaint investigations. The Board maintains on call, round-the-clock, a team of investigators in each of its six regional offices. The Pollution Response Program (PReP) Teams can be dispatched day or night to respond to pollution complaints. The rapid rise in pollution complaints --from 154 in 1970 to about 824 in 1977--is due, at least in part, to increased awareness on the part of the citizens of the State, and the fact that regional offices are located throughout the State. (View Graphs 12 and 13). The number of pollution complaints has levelled off somewhat in the last three years, and although the number of incidents may still seem large, damage to the environment has, in most cases, been mitigated by prompt containment and cleanup.

Permit Program

Section 402 of the Federal Water Pollution Control Act Amendments of 1972 (P.L. 92-500) requires that all point source (point source is defined as any discernible, confined, and discrete conveyance such as sewer pipe, culvert, tunnel, etc.) discharges of wastewater obtain a National Pollutant Discharge Elimination System (NPDES) Permit from the Environmental Protection Agency (EPA) or from a State authorized to implement the NPDES Permit Program. In accordance with this section, the Commonwealth of Virginia requested and was authorized to implement the NPDES Permit Program on March 31, 1975. In order to obtain this authorization, several changes were made in the State Water Control Law and the Board adopted Regulation #6. (View Graph 14) Thus, when a discharger obtains an NPDES permit from the Board he is complying with the State Law and the Federal Act. Since the authorization was received from EPA to administer the NPDES Permit Program, the Board has issued 79 major industrial NPDES permits and 100 major municipal permits. We have also issued 903 minor industrial and 815 minor municipal permits. (View Graph 15)

All the NPDES permits that were issued contained specific effluent limitations. If the permittee was unable to achieve the required limitations,the permit included interim effluent limitations and a compliance schedule for achieving the required future limitations. The permit also contained a requirement that the permittee submit to the Board by the 10th of each month a report which summarizes the permittee's self-monitoring program for the previous month. This report, which is known as the Discharge Monitoring Report or DMR, is tracked by the Board's staff to determine if the permittee is complying with the permit limitations.

In addition to the tracking program, the Board's staff performs periodic effluent sampling and testing of the permitted facilities. We also perform routine inspections of the wastewater treatment facilities and periodically evaluate the permittee's laboratory. All of the programs are intended to determine the acceptability of the permittee's self monitoring program and to verify the permittee's compliance with the NPDES permit.

As previously mentioned, all the NPDES permits that were issued contained effluent limitations. P.L. 92-500 required that specific effluent limitations be achieved by certain deadlines and the method for ensuring compliance with these deadlines was to incorporate the effluent limitations and, if applicable a schedule of compliance in the permittee's NPDES Permit.(View Graph 16) By July 1,1977, all point source dischargers were required to achieve either a minimum level of treatment or their effluent was to comply with the State's adopted Water Quality Standards. The more restrictive of these two requirements was the governing factor and these effluent limitations were included in the permit. For publicly owned treatment works (POTW), the minimum level of treatment was defined as secondary treatment and for industrial dischargers the minimum level of treatment was defined as the best practicable waste treatment technology (BPT).

By July 1,1983, P.L. 92-500 required that POTW's achieve the best practicable waste treatment technology (BPWTT) and industrial dischargers achieve the best available technology economically achievable (BAT).

P.L. 95-217 (Clean Water Act), which was approved in December 1977, significantly amended P.L. 92-500 and the deadlines for the POTWs and the industrial dischargers for achieving certain levels of treatment. (View Graph 17) P.L. 95-217 allows a case-by-case relaxation of the July 1, 1977 deadline for POTWs up to a maximum extension to July 1, 1983, in those instances wherein the July 1, 1977 deadline was not met through no fault of the POTW, such as insufficient grant funding. Private owners planning to connect to affected POTWs can get matching extensions. However, if a POTW owner wanted to have his NPDES permit modified to include the July 1, 1983 deadline, he must have requested this modification by June 26, 1978.

P.L. 95-217 now requires that industrial dischargers achieve by:

July 1, 1984 - Best Conventional Pollutant Control Technology (BCT) for Conventional Pollutants

July 1, 1984 - Best Available Technology Economically Achievable (BAT) for Toxic Pollutants

July 1, 1987 - Best Available Technology Economically Achievable (BAT) for Non-Toxic, Non-Conventional Pollutants

No-Discharge Certificate Program

The Board issues no-discharge certificates for treatment facilities utilizing recycle or reuse processes and for wastewater treatment systems that utilize land disposal as their ultimate method for disposing of the treated liquid waste. The no-discharge certificate program has been primarily involved with animal waste and the mechanism of land disposal that is utilized by the majority of these wastewater systems. For example, we have now issued 622 animal waste no-discharge certificates and 281 other no-discharge certificates. (View Graph 18) To give you some idea of the impact of the animal waste no-discharge certificate program, we estimate that the facilities permitted by these certificates have prevented the equivalent of the wastewater generated from approximately 628,000 people from entering State waters.

Sewerage Regulations

The State Water Control Law requires that most plans and specifications for sewerage systems be submitted to and approved by the State Department of Health (Department) and the Board. To assist in implementing this requirement, the Department and the Board adopted Sewerage Regulations effective February 1, 1977, that consisted of three sections: procedural regulations for submittal of and obtaining approval for plans and specifications; operational regulations; and a Manual of Practice. (View Graph 19). The regulations also establish procedures for obtaining a Certificate to Construct and a Certificate to Operate.

These regulations were adopted following many public meetings and public hearings and before adoption, they were concurred with by a select panel that consisted of representatives from the Virginia Municipal League, the Hampton Roads Sanitation District Commission, the State Water Control Board, the Virginia Association of Counties, the Consulting Engineers Council, the Virginia Society of Professional Engineers, and the State Department of Health.

The intent of the Sewerage Regulations was to provide the consulting engineers and the sewerage owners in the State with the general requirements of the Department and the Board and the procedures that the two agencies utilize in approving plans and specifications for sewerage systems. In fiscal year 1978, the Board will review about 253 sets of plans and specifications.

Pretreatment Program

A program area that has not received much emphasis thus far but one that has the potential of having a significant impact on the municipalities and the indirect industrial dischargers is the pretreatment program. When I use the term "indirect industrial discharger", I am referring to an industry that discharges its wastewaters into a publicly owned sewerage system. EPA has promulgated national pretreatment regulations which establish the roles and responsibilities of the Federal, State and local government and industry to implement the national pretreatment standards controlling non-domestic discharges which are "incompatible" (pass through or interfere) with the treatment processes of publicly owned treatment works. (View Graph 20)

The pretreatment regulations will apply to municipalities whether or not the POTW is federally funded. All local authorities operating publicly owned treatment works with total flows of 5 million gallons per day and which receive industrial waste subject to the national pretreatment standards will be required to establish pretreatment programs. The establishment and enforcement of the local pretreatment program will be a condition of the NPDES Permit. Also, where needed, establishment of a pretreatment program will be a condition of any new construction grant. Publicly owned treatment works with flows less than 5 million gallons per day will be required to develop pretreatment programs where special problems warrant.

401 Certification Program

Section 401 of P.L. 92-500 requires that any applicant requesting a Federal license or permit to conduct any activity which may result in any discharge into navigable waters provide the licensing or permitting agency a certification from the State in which the discharge occurs stating that the discharge will comply with the State Water Quality Standards or any applicable effluent limitations. (View Graph 21) This certification process involves projects such as power plants, docks, dredging operations, alterations to streams, refineries, dams, etc. Most of the projects are in the Tidewater area of the State and involve the Corps of Engineers. However, the 401 Certification is applicable throughout the State and could involve the Federal Power Commission, the Nuclear Regulatory Commission, etc. The State Water Control Board issues the 401 Certificates in Virginia and during fiscal year 1978, we estimate that approximately 400 will be processed.

Summary

In closing, I would like to leave you with this thought which I believe illustrates the success of the Commonwealth's water pollution control and abatement programs. (View Graph 22). In 1975, 91.6% of Virginia's over 27,140 stream miles met "305(b) criteria" (swimmable, fishable waters); in 1977, 94.8% met the same criteria; and by 1983, we estimate that 99.7% of the stream miles in the State will meet these standards. I believe these figures speak for themselves.

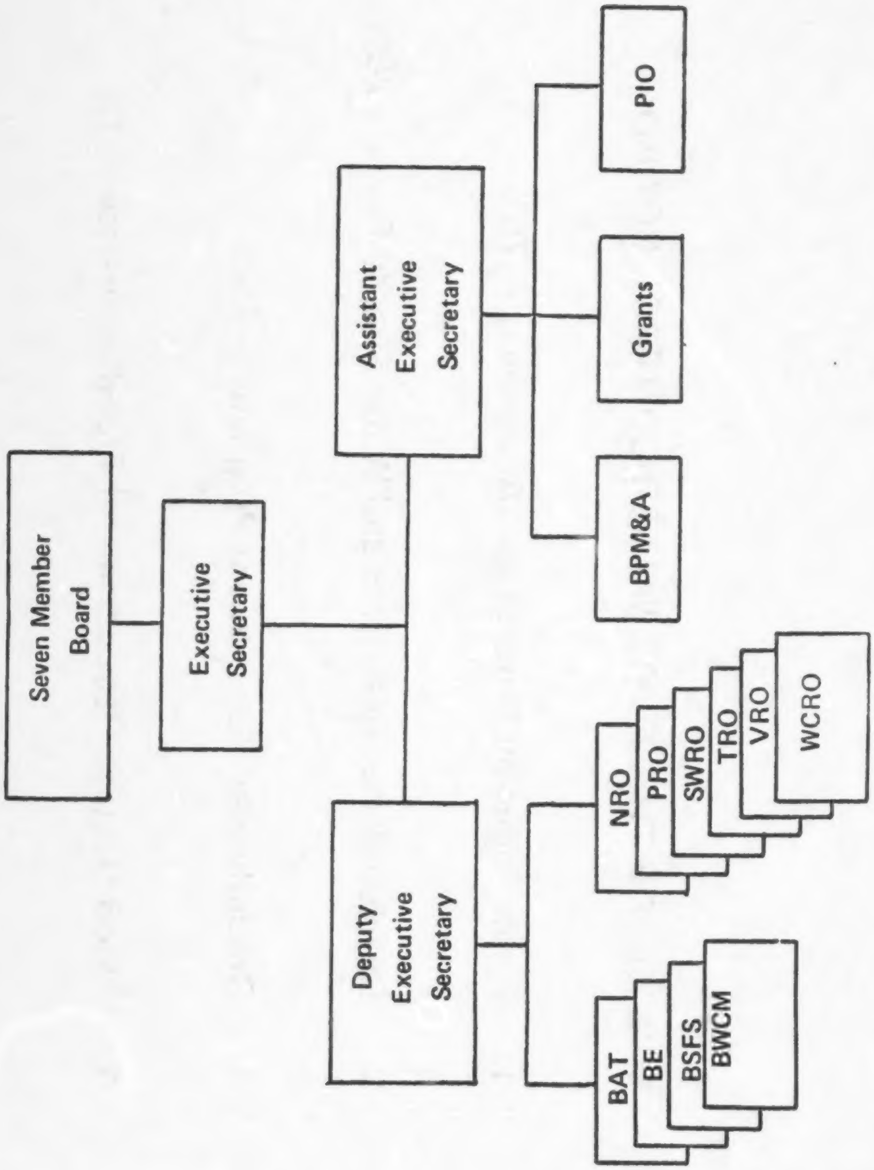
VIEW GRAPH 1

MISSION OF THE STATE WATER CONTROL BOARD

1. Water Pollution Control and Abatement – 75%
2. Develop Water Resources and Water Supply Plans – 20%
3. Groundwater Management Programs – 4%
4. Flood Damage Reduction and Flood Insurance – 1%

VIEW GRAPH 2

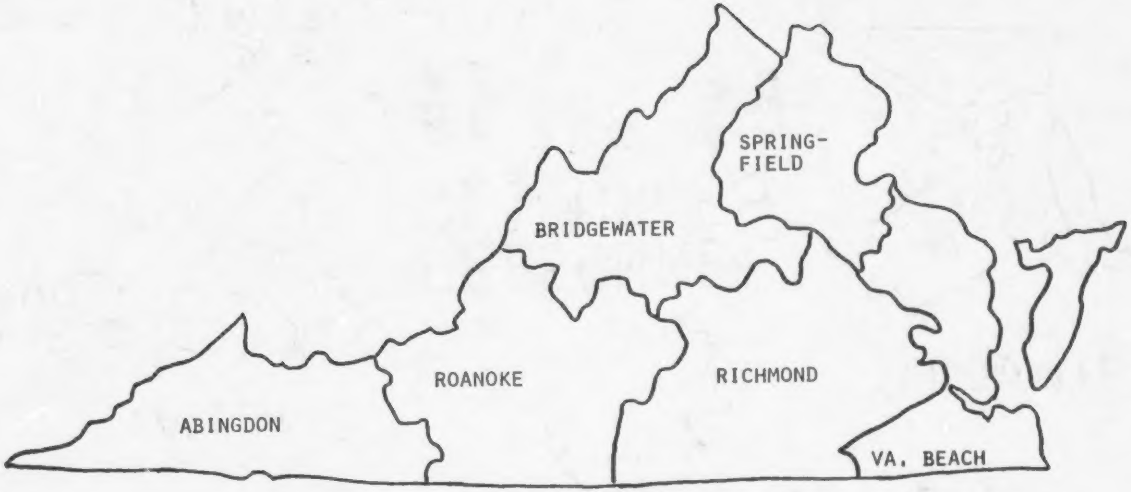
AGENCY ORGANIZATIONAL CHART



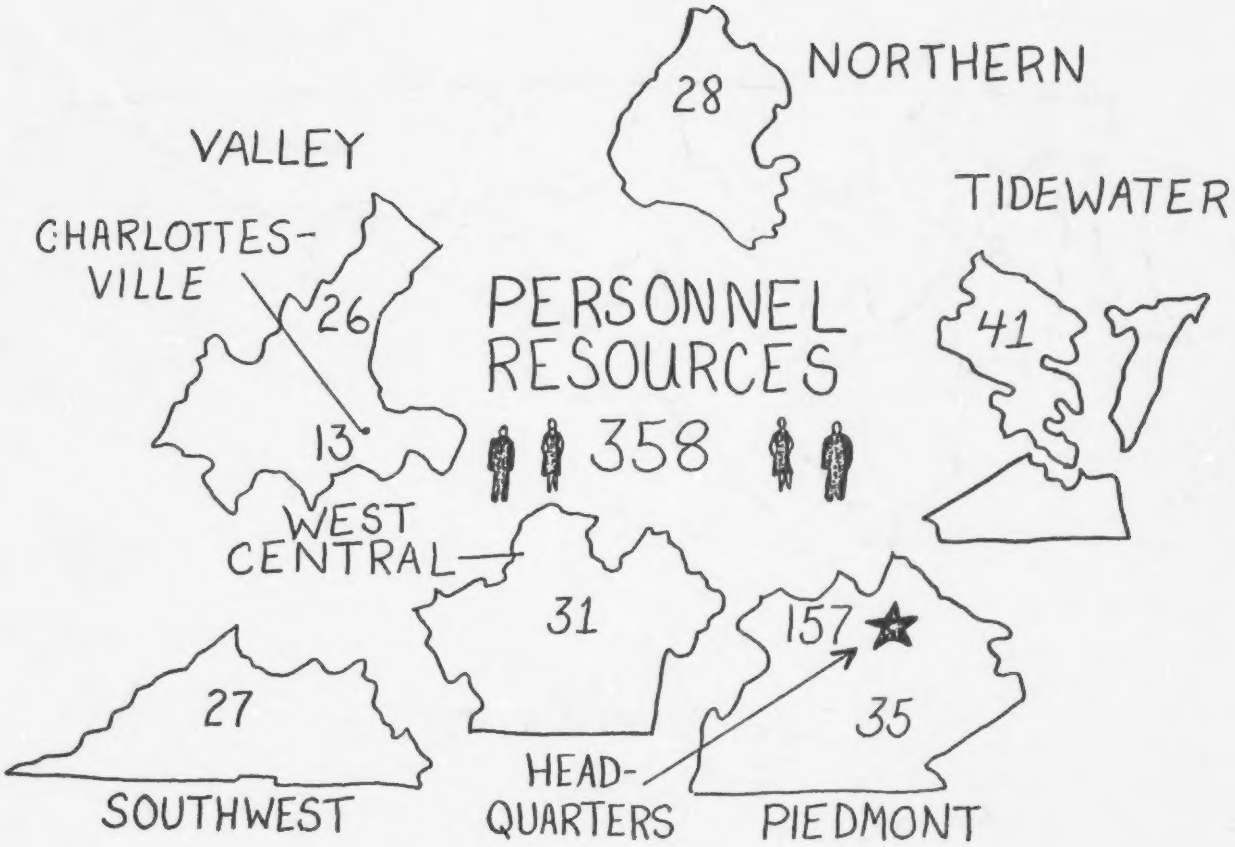
VIEW GRAPH 3

REGIONALIZATION

SWCB

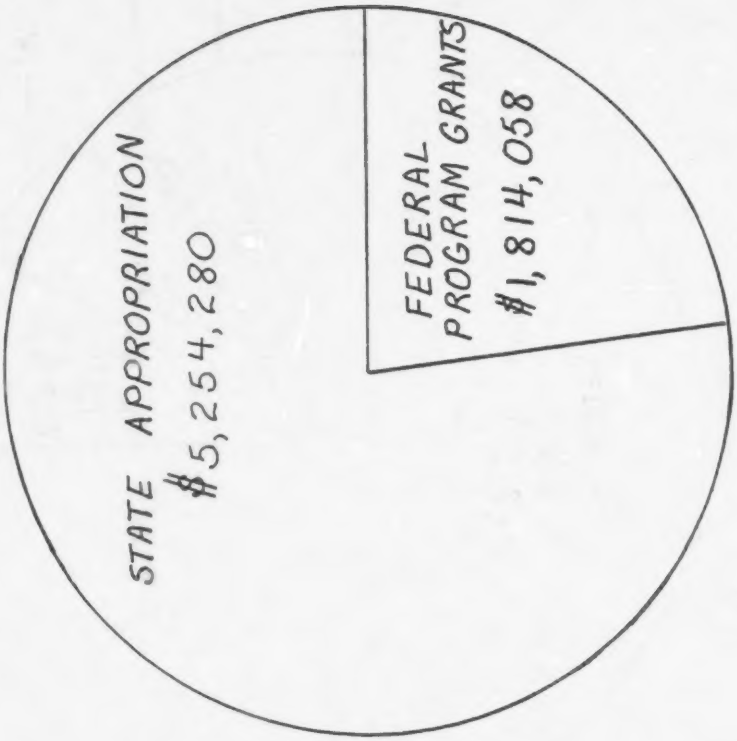


VIEW GRAPH 4

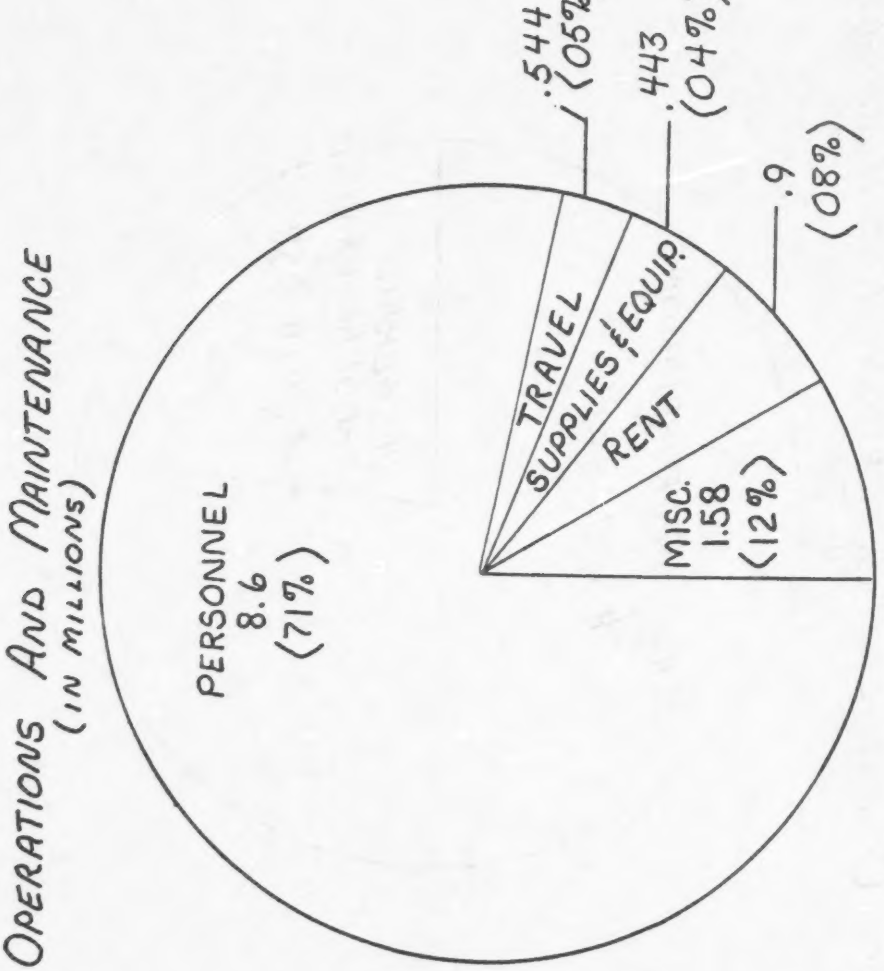

PERSONNEL RESOURCES
358
NORTHERN
28
VALLEY
CHARLOTTES-VILLE
26
13
TIDEWATER
41
WEST CENTRAL
31
SOUTHWEST
27
HEAD-QUARTERS
157
35
PIEDMONT

VIEW GRAPH 5

SOURCES OF AGENCY'S OPERATIONS AND MAINTENANCE
FY-78

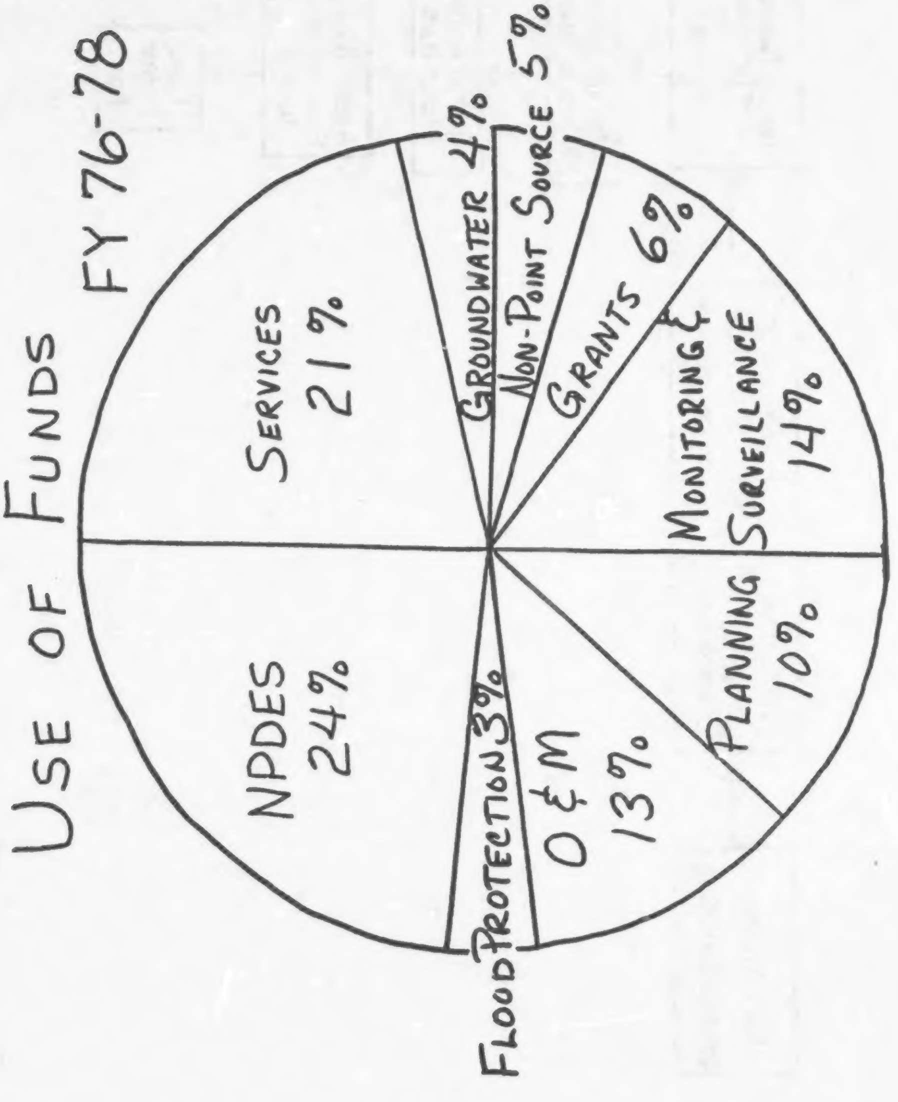


VIEW GRAPH 6

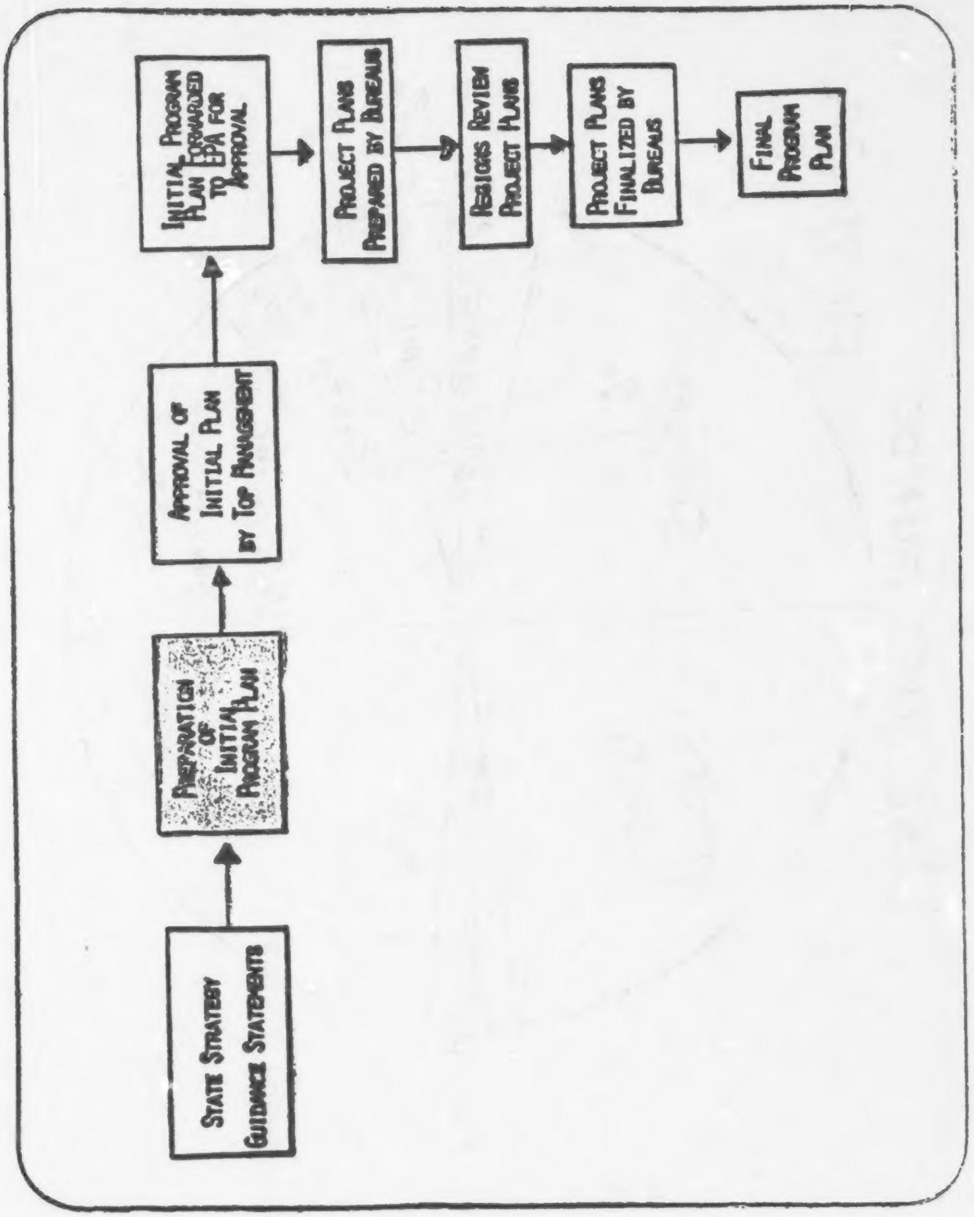


OPERATIONS AND MAINTENANCE
(IN MILLIONS)

VIEW GRAPH 7


USE OF FUNDS FY 76-78
SERVICES 21%
GROUNDWATER 4%
NON-POINT SOURCE 5%
GRANTS 6%
MONITORING & SURVEILLANCE 14%
PLANNING 10%
O & M 13%
FLOOD PROTECTION 3%
NPDES 24%

VIEW GRAPH 8


State Strategy Guidance Statements
Preparation of Initial Program Plan
Approval of Initial Plan by Top Management
Initial Program Plan Forwarded to EPA for Approval
Project Plans Prepared by Bureaus
Regions Review Project Plans
Project Plans Finalized by Bureaus
Final Program Plan

VIEW GRAPH 9

GRANT APPROPRIATIONS & PROPOSED ALLOCATIONS UNDER P.L. 95-217

FISCAL YEAR	NATIONAL	VIRGINIA
1978	$4.5 Billion	$88,209,000
1979	4.5 Billion	88,209,000
1980	4.5 Billion	88,209,000
1981	4.5 Billion	88,209,000
1982	4.5 Billion	88,209,000
TOTALS:	$22.5 Billion	$441,045,000

VIEW GRAPH 10

BREAKDOWN BY AREA OF GRANT FUNDS NEEDED

1978 DOLLARS

	GRANT FUNDS NEEDED
o NORTHERN VA PROJECTS (TO FULFILL VA'S COMMITTMENT TO CLEAN UP THE POTOMAC RIVER)	$ 84,433,000
o HRSD'S PROJECTS (TO UPGRADE TREATMENT & CONSTRUCT NEW PROJECTS)	$153,350,000
o RICHMOND AREA PROJECTS (TO COMPLETE JAMES RIVER CLEAN UP)	$ 218,139,000
o REST OF THE STATE'S PROJECTS (TO CONSTRUCT NEEDED TREATMENT FACILITIES)	$346,151,500
TOTAL:	$ 802,073,500

VIEW GRAPH 11

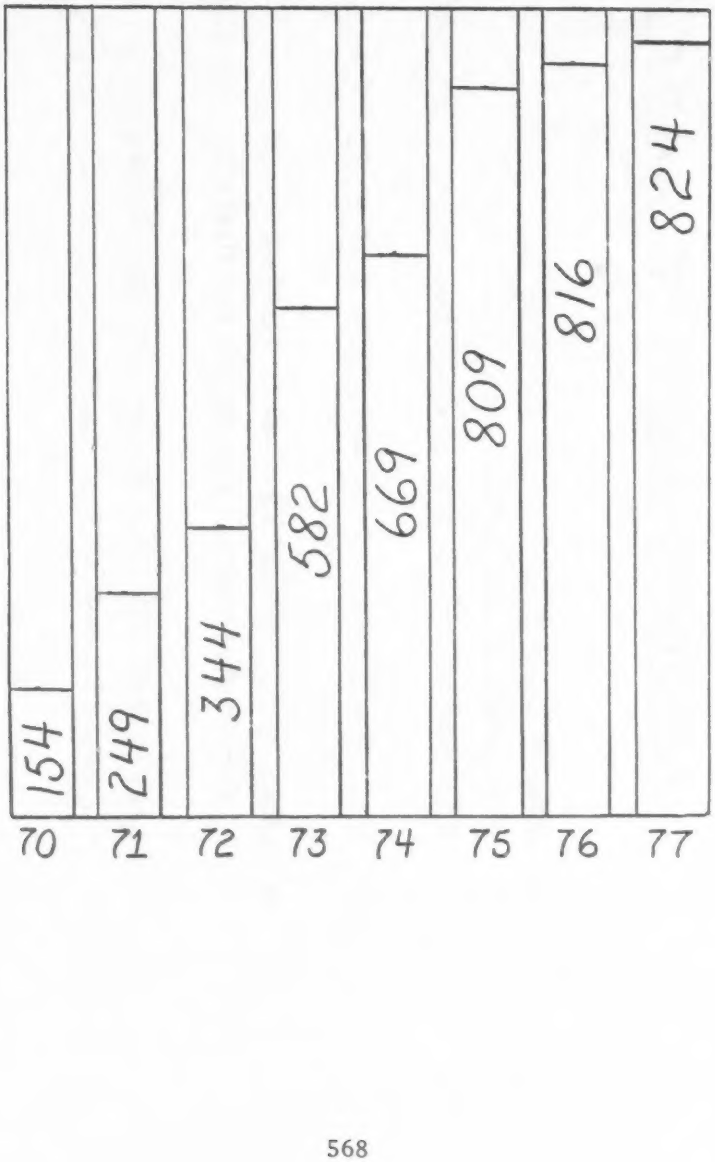
ANTICIPATED SHORTAGE OF GRANT FUNDS

o GRANT FUNDS NEED TO COMPLETE PLANNING & CONSTRUCTION OF SEWERAGE FACILITIES (1978 DOLLARS)	$ 802,073,500
o LESS FEDERAL APPROPRIATION FOR FY78	(- 88,209,000)
o LESS FY 79 thur 82 FUNDS AUTHORIZED (FUNDS AUTHORIZED - NOT APPROPRIATED)	(-352,836,000)
o ANTICIPATED SHORTAGE OF GRANT FUNDS (BASED ON 1978 COST) - - - - - - - -	$ 361,028,500

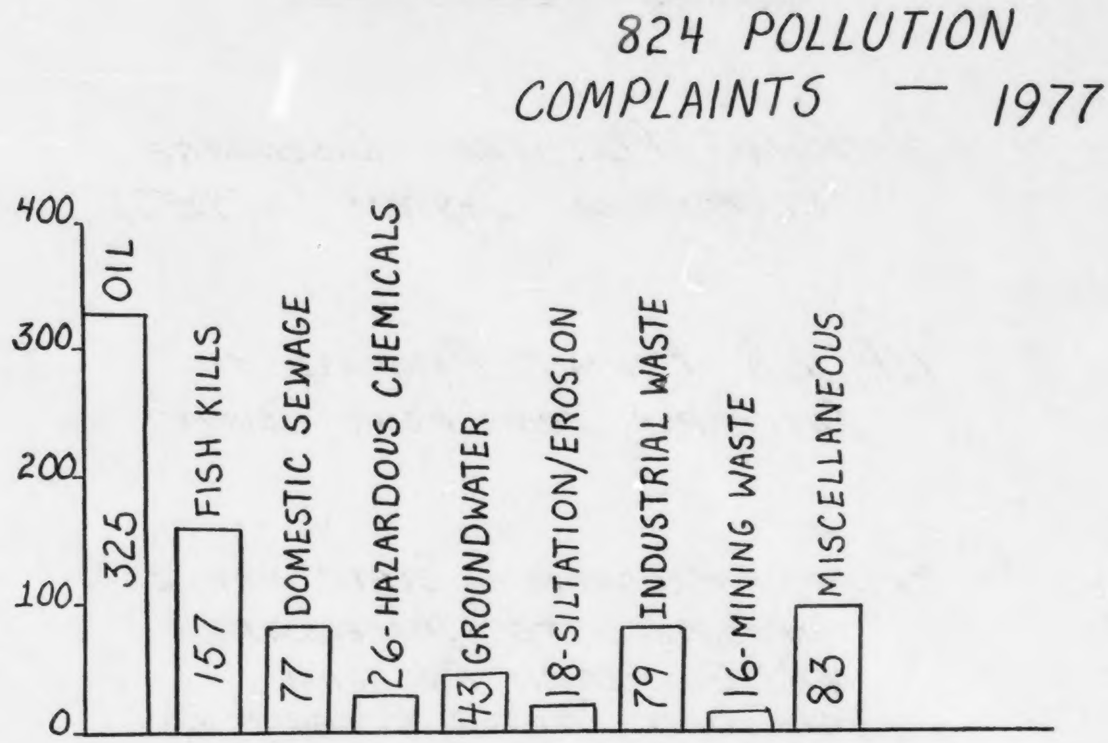
* INFLATION WILL INCREASE THESE AMOUNTS

VIEW GRAPH 12

POLLUTION COMPLAINTS



VIEW GRAPH 13



PERMIT PROGRAM

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM (NPDES)

NPDES PERMIT REQUIRED TO DISCHARGE FROM POINT SOURCE

FEDERAL PROGRAM – STATE MAY BE AUTHORIZED TO IMPLEMENT NPDES PERMIT PROGRAM PROVIDED STATES PROGRAM IS COMPARABLE TO FEDERAL PROGRAM

REGULATION 6 – VIRGINIA NPDES PERMIT PROGRAM

NPDES PERMITS ISSUED

	INDUSTRIAL	MUNICIPAL
MAJOR	79	100
MINOR	903	815

P.L. 92-500
EFFLUENT REQUIREMENTS

JULY 1, 1977

MINIMUM LEVEL OF TREATMENT
  PUBLICLY OWNED TREATMENT WORKS (POTW.
    SECONDARY TREATMENT
  INDUSTRIAL DISCHARGERS — BEST
    PRACTICABLE WASTE TREATMENT
    TECHNOLOGY (BPT)
WATER QUALITY STANDARDS

JULY 1, 1983

POTW — BEST PRACTICABLE WASTE
  TREATMENT TECHNOLOGY (BPWTT)
INDUSTRIAL DISCHARGERS — BEST
  AVAILABLE TECHNOLOGY
  ECONOMICALLY ACHIEVABLE (BAT)

P.L. 95-217
EFFLUENT REQUIREMENTS

PUBLICLY OWNED TREATMENT WORKS (POTW) -

CASE-BY-CASE EXTENSION OF THE JULY 1, 1977, DEADLINE TO JULY 1, 1983, TO ACHIEVE SECONDARY TREATMENT OR THE WATER QUALITY STANDARDS

DEADLINE TO REQUEST EXTENSION -
JUNE 26, 1978

INDUSTRIAL DISCHARGERS

JULY 1, 1984 - BEST CONVENTIONAL POLLUTANT CONTROL TECHNOLOGY (BCT) FOR CONVENTIONAL POLLUTANTS

JULY 1, 1984 - BEST AVAILABLE TECHNOLOGY ECONOMICALLY ACHIEVABLE (BAT) FOR TOXIC POLLUTANTS

JULY 1, 1987 - BEST AVAILABLE TECHNOLOGY ECONOMICALLY ACHIEVABLE (BAT, FOR NON-TOXIC NON-CONVENTIONAL POLLUTANTS

NO-DISCHARGE CERTIFICATE PROGRAM

REUSE OR RECYCLE SYSTEMS

ANIMAL WASTE

TOTAL ISSUED - 903

SEWERAGE REGULATIONS

PROCEDURAL REGULATIONS

OPERATIONAL REGULATIONS

MANUAL OF PRACTICE

PLANS AND SPECIFICATIONS
REVIEWED IN FY 78 – 253

PRETREATMENT PROGRAM

EPA REGULATIONS

POTW $\geq$ 5 MGD RECEIVING INDUSTRIAL WASTE SUBJECT TO PRETREATMENT STANDARDS REQUIRED TO HAVE PROGRAM

POTW $<$ 5 MGD REQUIRED TO HAVE PROGRAM WHERE PROBLEMS WARRANT

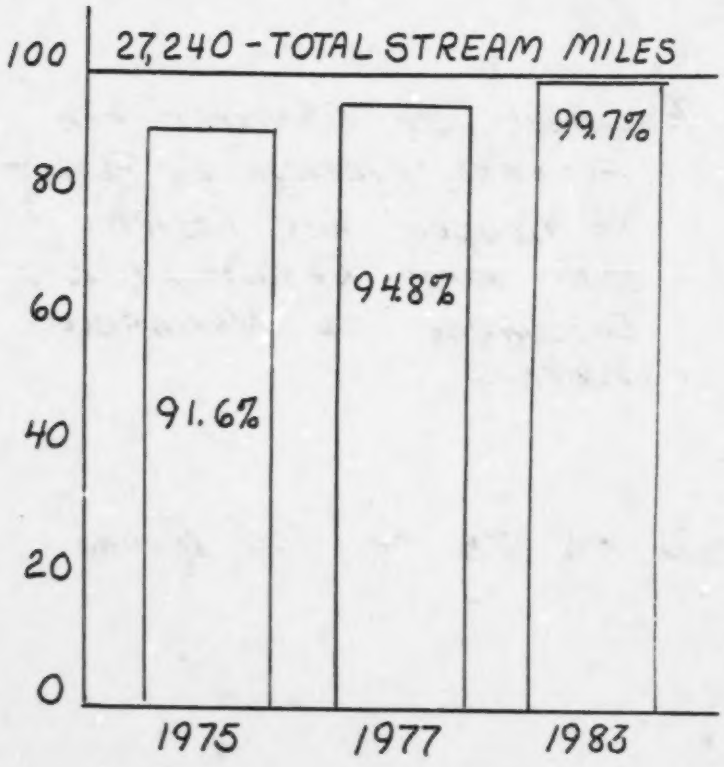
401 CERTIFICATION PROGRAM

REQUIRED BY APPLICANT FOR FEDERAL LICENSE OR PERMIT TO CONDUCT ANY ACTIVITY THAT MAY RESULT IN ANY DISCHARGE TO NAVIGABLE WATERS

IN FY 78 ~ 400 ISSUED

VIEW GRAPH 22

% OF VIRGINIA'S WATERS MEETING 305(b) CRITERIA



WATER SUPPLY AND WASTEWATER MANAGEMENT PLANNING IN THE WASHINGTON, D.C., METROPOLITAN AREA*

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and
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Introduction

Water supply and wastewater management planning in the Washington Metropolitan Area, which includes approximately three-quarters of the Potomac basin's population of four million, is fragmented among agencies of two states (Maryland and Virginia), the District of Columbia, ten counties and cities with general-purpose local government jurisdiction, two multi-county authorities, at least two federal agencies with direct and/or indirect responsibilities, the Metropolitan Washington Council of Governments, and the Interstate Commission on the Potomac River Basin (ICPRB). From this fragmented and confusing milieu, the authors will attempt to provide some light on the background for planning, on where we are now with current planning activities, on ICPRB's role, and perhaps most useful of all, highlight some planning tools which are currently being employed in the Washington Metropolitan area and may be useful in Japan and other parts of the world.

Wastewater Management

Water pollution and its control have had a long history in the Washington Metropolitan Area. Some of these major events are listed in A Clean-Up Chronology compiled by the ICPRB and published in The Washington Post newspaper in 1977 (copy attached).[1]

Most of the wastewater treatment capacity in the Washington Metropolitan area is provided by the 309 mgd (13,500.2 L/s) regional Blue Plains treatment plant in the District of Columbia. Under a regional memorandum of understanding in 1970, reaffirmed by an agreement in 1971, the District of Columbia's share of that capacity is 43.7 percent and the allocation to the Washington Suburban Sanitary Commission (WSSC), which provides sewage as well as water service in the Maryland suburbs, is 49.6 percent. The remaining 6.7 percent is allocated to Virginia suburban counties and cities and serves only a small portion of their needs.

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*Prepared for the United States/Japan Conference on Sewage Treatment Technology, Washington, D.C., November 2-3, 1978.

[1] A Clean-Up Chronology, Washington Post, June 16, 1977.

In 1973, controversy about allocation of Blue Plains capacity, raw sewage overflows, and the responsibility for sludge disposal led to a suit in the U.S. District Court for the District of Columbia. A consent decree signed in June 1974 included a schedule for settlement of the issues under the jurisdiction of the District Court. The only remaining long-term issue to be settled is the question of allocation of sites and the determination of the methods for disposal of sludge from Blue Plains after 1979.

In addition to capacity in the Blue Plains regional plant, WSSC has three sewage treatment plants in the Maryland suburbs, the largest of which has a capacity of 30 mgd (1,300 L/s). Most of the Virginia suburban sewage will soon be treated in five plants, with a combined design capacity of about 150 mgd (6,600 L/s), all but one of which will serve more than one local government jurisdiction. The Northern Virginia subregionalization was achieved primarily by mandates from the Virginia State Water Control Board. One of these plants, the Upper Occoquan Sewage Authority (UOSA) advanced wastewater treatment plant, started operating in July of this year, thus eliminating several smaller and less efficient plants. UOSA was established to reduce point source pollutants, particularly phosphorus, in the watershed of the Occoquan Reservoir, the major source of water supply for the Virginia suburbs.

The areawide waste management planning in the Washington Metropolitan area under Section 208 of the Federal Clean Water Act has been conducted by the Water Resources Planning Board of the Metropolitan Washington Council of Governments. The draft 208 plan was completed early in 1978, but several of the local governments objected to certain provisions. Two major points of disagreement were: (1) capacity-sharing in treatment plants for sewage generated by future growth and, (2) sludge treatment or disposal sites. These disagreements were not cured by last minute efforts before the June 30, 1978, deadline for delivery of the initial 208 plan to the states for certification to the U.S. Environmental Protection Agency for approval. Therefore, they remain for solution in the continuing planning process and the U.S. District Court.

In the institutional analysis of the 208 plan, the Water Resources Planning Board adopted an option that existing agencies and programs assume responsibility for implementing the endorsed strategies and other elements of the plan, with plant sizings to be based on regional needs and more interjurisdictional contracts to establish the increased interdependence.

In summary, one is compelled to conclude that in the Metropolitan Washington area, which contains over three-quarters of the Potomac basin's four million population, wastewater management is fragmented but considerable regionalization does exist under interjurisdictional contracts. The latter probably will be extended, but slowly and incrementally, partly influenced by voluntary actions of local governments and partly under pressure from state water pollution control agencies, U.S. Environmental Protection Agency sanctions, and the courts.

ICPRB's Role in Water Quality Management Planning

One of the major functions of ICPRB is to mediate disputes arising among the states and interstate disputes among local jurisdictions. To facilitate this in the case of the current capacity sharing dispute among the local jurisdictions, the ICPRB is developing the Sewer Expansion Simulation (SES), a planning tool which allows the very rapid creation and comprehensive evaluation of technically and legally feasible alternatives for sewage disposal.

Such a tool is required for many reasons. One is the fact that minimizing cost is not the only, nor even the dominant, factor affecting the resolution of the capacity sharing issue. The political "saleability" of the location of new and expanded facilities is also an important factor. Political decision makers must, therefore, have a hand in designing alternatives for the simple reason that information concerning political "saleability" is not available to the engineer or planner. The politician must be provided with the ability to create or influence the development of alternatives. A second requirement is to reduce the cost of the planning effort itself. Resolution of the issues will require the evaluation of a very large number of alternatives. If this process is automated, significant savings in time and money can be achieved.

A third requirement concerns the nature of the decision making process itself. The political compromises necessary to resolve the problem will likely take place at political meetings of several hours duration. The ability to evaluate the technical feasibility and cost implications of such compromises during the progress of such meetings should be highly valuable.

Sewer Expansion Simulation

The Sewer Expansion Simulation meets these requirements. It is an interactive computer program which guides its user in the preparation of alternatives which provide conveyance and treatment capacity sufficient for existing and projected future development. It points out when additional capacity is required so that it can be sequenced to reduce the present value (PV) of capital expenditures. It also provides for efficient operation of facilities through the planning period in order to reduce operation and maintenance (O&M) costs. Further, the program ensures that every alternative created will not violate water quality standards. It does so by requiring that the user select sufficiently high treatment levels at sewage treatment plants. When an alternative is complete, the program provides the user with tabular output which completely describes the alternative in terms of facilities, their capital and operating costs for each year in the planning period, and the PV of capital, O&M, and total cost. The SES has been implemented for one county in the metropolitan area, Montgomery County, Maryland. This was accomplished in cooperation with the Montgomery County Planning Board of the Maryland National Capital Park and Planning Commission. Work is continuing to implement the SES for the entire metropolitan area.

The heart of the SES is a network model which simulates the operation of the sewer system. A schematic of this model is shown in Figure 1. This section will describe the relationship of the parts of the model to the real world, and the source of data for the parameters of the model. The circles in Figure 1 will be referred to as nodes and the lines joining the circles as arcs. Flows in the arcs represent average daily sewage flows in sewers, at treatment plants, or into receiving streams. All flows originate at the leftmost node in the Figure (labled the supersource), and terminate at the rightmost node (labeled the supersink). No flow is lost or gained at any other nodes.

The nodes in the column labeled sources correspond to segments of the sewered area. They are called sewersheds because they are analogous to watersheds except that the drainage is by sewers, not streams. The source nodes also correspond to the point in the sewer system at the downstream point of the sewershed. All sewage generated in the sewershed drains to this point. The sewersheds are determined using maps of the existing sewer system and topographic maps of currently unsewered areas which are expected to be sewered. In the Washington area they range in size from about one to about ten square miles (3-30 km^2).

In order to estimate the future flows from each sewershed, population and employment estimates are made for each sewershed for a set of future years. The Montgomery County application makes estimates for 1985 and 1995. Ability to predict future population and employment accurately for small areas determines the minimum size of the sewersheds. Future flows are then determined by taking the existing flow and adding (or subtracting) so much flow per capita and per employee for the increase (or decrease) in population and employment in the sewershed. A small amount of infiltration is also added for each additional person. Existing flows are determined from flow monitoring information. Per capita and per employee water use above each monitoring point is determined by dividing residential water use by existing population (per capita) and all other water use by existing employment (per employee). Infiltration or water loss is estimated by subtracting total water use above a monitoring point from measured flow. When there is more than one sewershed above a monitoring point, flows are apportioned on the basis of population and employment. When the model simulates the operation of the system it attempts to force flows equal to the flow projections for the year it is simulating through every arc between the supersink and a source node.

These flows must leave the source nodes through the arcs between the source nodes and the intermediate nodes and STP (sewage treatment plant) nodes. These arcs represent the sewers and force mains in the system (see Figure 1). Each such arc represents a stretch of sewer, which has a maximum capacity to carry flow. This is determined from the size, slope, and condition of the sewer. Because flows in the sewers are variable, capacity on each arc is given in terms of capacity for average daily flow. This is empirically related to the maximum capacity by a peak to average ratio. The ratios used in Montgomery County are given in Table 1.

Because the arcs between source, intermediate, and STP nodes represent real facilities, they have associated operating and maintenance costs which are dependent on flow. These costs are represented by linear cost functions as shown schematically in Figure 3. Note that costs for sewers are much less dependent on flow than costs for force mains because of the energy required for pumping.

Sink nodes represent discharge points on receiving waters. Arcs between STP and sink nodes represent sewage treatment facilities. Like the arcs representing sewers and force mains, these have associated capacities and costs. Arcs between the sink nodes and the supersink represent the receiving waters. These waters have an assimilative capacity which depends on factors such as temperature, velocity, flow, and applicable standards. Given the concentration of pollutants in the discharge of a sewage treatment plant, the assimilative capacity of the receiving water can usually be expressed as a flow of effluent from the STP. This is the capacity assigned to sink-supersink arcs.

A flowchart for the interactive program is given in Figure 2. Initially, the program sets up the network model using the existing sewage system. It then reads the current flow data and attempts to route existing flows from the sewersheds (source nodes) through the sewer network, sewage treatment plants and receiving waters without violating any capacity constraints. The Out of Kilter Algorithm (OKA), which is utilized to perform this routing, finds the routing which minimizes operating costs for that year. Assuming a feasible routing (which corresponds to an operating strategy) can be found, the program revises the flow requirements on the arcs between the surpersource and the source nodes to correspond to the flow projections for the next period. It then attempts to route these flows through the sewer system.

If no feasible routing can be found the program prints the locations of the bottlenecks in the system and list the sewersheds which contribute flows above those bottlenecks. It then requests user input as to what facilities are desired to correct the problem. The user may input sewers, force mains, or land treatment facilities. The program then requests other information such as lengths, capacities, treatment levels (for treatment plants) and flows or assimilative capacities of receiving waters. The capital and operating costs for these facilities are computed from cost curves and displayed at the terminal; the user may then implement them in the network model. Given these changes, the program again attempts a routing, progressing to the projections for the next period if a feasible routing is found, and asking for additional system improvements otherwise.

After a feasible routing for the last period has been found the program produces a capital improvements program based on the facilities which have been input. Costs are spread evenly for each planning period and a present value analysis is performed. The user can also display the optimal routings used to determine operating costs. A portion of the interactive output from a run of the program is given in the Appendix. Input and output formats can easily be altered to meet the needs of individual users.

Water Supply Management

The three major water utilities in the Metropolitan Washington Area are U.S. Army Corps of Engineers' Washington Aqueduct Division (COE), the Washington Suburban Sanitary Commission (WSSC), and the Fairfax County Water Authority (FCWA). Together they serve a population of almost three million people.

Washington was still a small city served by water from numerous springs, shallow wells and small streams when the U.S. Congress in 1852 authorized the COE to study means for supplying ample water to the city. The COE plan for bringing water into the city from the Potomac River at Great Falls, ten miles (16.09 km) to the northwest, was approved and the construction of the gravity conduit and small storage reservoirs within the District of Columbia was completed in 1863. Many of the original structures are still being used today.

As the metropolitan area grew beyond the District of Columbia, the Washington Aqueduct Division (WAD) extended its system to supply part of the suburbs in Northern Virginia. It now serves about 1.1 million people from two intakes on the Potomac River.

The second of the three major water supply utilities, the WSSC, was created by the Maryland State Legislature in 1918 to acquire and integrate several small municipal systems in the Maryland suburbs of the District of Columbia. Sediment problems and the need for more storage capacity led to construction of two major reservoirs on the Patuxent River northeast of the District of Columbia, which were completed in 1942 and 1952. Population growth in the Maryland suburbs was so rapid after World War II that WSSC also completed an intake and filtration plant on the Potomac upstream from Washington in 1951. The latter now serves most of the 1.2 million people served by WSSC in the Maryland suburban counties of Montgomery and Prince Georges. A water supply task force made up of elected officials of the county governments, technical staff from WSSC and county agencies, and a citizen advisory committee participated in a two-year planning study which ended in the spring of this year (1978). The recommendations included (1) construction of a small reservoir on a tributary of the Potomac upstream of the WSSC intakes to augment Potomac low flows for water supply and also to provide recreational and sediment control benefits, (2) construction of a pipeline between the WSSC Potomac River and Patuxent River filtration plants so that water could be transported in either direction to increase the reliability of either supply source, and (3) implementation of the concept of water demand management, also called drought management. The latter would involve planned restrictions on water use, such as lawn irrigation, during the relatively short and infrequent periods of peak water use in extreme droughts of low probability.[2] All of these recomendations are progressing through additional preliminary planning stages to verify technical and financial feasibility prior to implementation.

The third major water utility, the FCWA, was created in 1957 by Fairfax County, the largest of the Virginia counties in the Metropolitan Washington Area, under an enabling act of the Virginia State Legislature. FCWA initially acquired and gradually integrated numerous small private and public water systems, most of which were served by ground water from wells. In the mid-1960's, FCWA acquired the facilities of a private water company serving the city of Alexandria which included a reservoir on Occoquan Creek, a tributary of the Potomac estuary downstream from Washington. The Occoquan Reservoir is the primary source of water for the population of 640,000 people now served by FCWA within Fairfax County and a portion of Prince William County, Virginia.

Continued growth in the Northern Virgnia suburbs has led to independent efforts by FCWA to better manage its Occoquan Reservoir supply[3], reduce peak demands by one of the first water rate schedules in the U.S. with peak summer demand surcharges, and plan for a new intake and filtration plant on the Potomac upstream from Washington. Site preparation for the latter started in July 1978.

This brief description demonstrates that both of the suburban utilities serving almost two-thirds of the Metropolitan Washington population are taking somewhat independent steps to increase their ability to serve their consumers. Both WSSC and FCWA will be relying more and more on the Potomac to serve future population growth. The COE is serving a more stable population but is faced with declining low flows in the Potomac due to increased withdrawals by others. This problem led to an agreement to share the Potomac low flows according an equitable formula among the three utilities, the governments of the States of Maryland and Virginia, the District of Columbia, and the Federal Government.[4] Yet to be settled is the amount of flow which will be allowed to flow by the intakes into the estuary for protection of water quality and fish and wildlife.

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[2] Bi-County Water Supply Study-Montgomery and Prince Georges Counties, Maryland, prepared by Technical Advisory Group, April 1978.

[3] Assessment of the Current Occoquan Water Supply Situation, Hirsh, R. (USGS), J. Schaake (NWS), and D. Sheer (ICPRB), ICPRB, October 25, 1977.

The COE civil works functions include comprehensive water resources planning and regulation of the navigable waterways throughout the U.S. Only in the Washington area does it function as municipal water supply utility. The most comprehensive water resources planning study in the Potomac basin was completed by the COE in 1963.[5]

The 1963 plan included 16 major multiple-purpose reservoirs, primarily to provide storage for augmenting Potomac River flow for water supply and water quality enhancement in the Metropolitan Washington Area, and numerous other projects for control of floods, erosion and sediment control in the basin. Only one of the reservoirs is being constructed (Bloomington, upstream from Washington on the Potomac North Branch, which is scheduled for completion in 1981), or is likely to be constructed.

This is due to strong opposition from people in the areas which would be affected by additional reservoirs and pressure from national conservation and environmental organizations. For example, in 1977, in the Corps discontinued preliminary design studies of the proposed Verona Reservoir in Virgnia on the Shenandoah River, the largest tributary of the Potomac, due to local and state opposition.

Current Corps of Engineers Potomac basin water supply planning is confined to a restudy of future needs and alternatives for Metropolitan Washington water supply under a 1974 Congressional authorization[6]. This authorization specifically directed the Corps to assess with a pilot plant the feasibility of the alternative of utilizing the uper Potomac tidal estuary as a water supply source.[7] Alternatives to additional upstream reservoirs and development outside of the Washington area (both violently opposed in the areas where they would be located) are being given the most intensive consideration at the present time. These alternatives include raw (untreated) water and finished water (treated) water interconnections among the three major water utilities and water conservation and demand management to reduce peak water use during periods of dry weather and low stream flow.

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[4] Potomac River Low Flow Allocation Agreement. U.S.A., Maryland, Virginia, District of Columbia, WSSC and FCWA, January 11, 1978.

[5] Potomac River Basin Report, U.S. Army Engineer District, Baltimore, Maryland, February 1963.

[6] Water Resources Development Act of 1974, Public Law 93-251, Section 85, March 7, 1974.

ICPRB's Role in Water Supply Management and Planning

As in the case of water quality, ICPRB attempts to assist in mediating disputes. It also serves as a source of information and technical assistance to the states, localities, and the Federal Government. ICPRB studies and reports have been a major factor in renewed interest in both raw and finished water interconnections linking the three major sources of water supply in the Metropolitan area.[8,9] These reports pointed out that by using such interconnections to operate the supplies in an integrated manner, the effective yield of the local reservoirs could be sustantially increased (in fact doubled), and that substantial cost savings could be achieved by integrating the operation and service areas of the major utilities.

ICPRB has cooperated closely with a detailed investigation of possible raw water interconnections to provide additional flexibility for such operations. This work has been performed for the Corps of Engineers by a private consultant.[10] In addition, the Corps funded an ICPRB study of finished water interconnections to achieve the same purpose.[11] This study involved implementing water distribution system models for all major water suppliers in the Metropolitan area, and testing the feasibility of operating strategies to conserve water in the local reservoirs whenever possible. The study was to identify those new facilities necessary to achieve such operation.

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[7] The Potomac Estuary: A Potential Water Supply, ICPRB, April 1978.

[8] A Prespective on the Washington Metropolitan Area Water Supply Problem, Sheer, D.P., et al, ICPRB, May 1977.

[9] More Water for Less Cost in the Washington Metropolitan Area, Sheer, D.P. and P.W. Eastman, ICPRB, April 16, 1978.

[10] The Optimization of Raw Water Interconnections for the Washington, D.C., Metropolitan Area, Draft Report, GKY and Associates, Inc., July 1978.

[11] Draft Report of the Metropolitan Area Finished Water Interconnection Study, ICPRB for the U.S. Army Corps of Engineers, Baltimore District, July 1978.

Surprisingly, this study found that such operation could be achieved without the construction of any new drought related facilities. The most severe periods of droughts occur in the fall rather than during summer. The water supply systems are designed to meet peak daily demands in the summer, some 60 percent larger than the average daily demand over the year. By using some of this peaking capacity off-peak to conserve water in the reservoirs (taking additional water from the Potomac both in anticipation of and after a drought), and by using some of the peaking capacity to withdraw some of the water thus saved from the local reservoirs during the drought, water supply requirements can be met through the year 2000 or beyond.

In addition to its work in water supply planning, the ICPRB provided technical assistance to the Fairfax County Water Authority, helping it to manage the crisis caused by the drought of 1977. During the late summer and fall of that year the FCWA's reservoir, the only source of water for some 650,000 people in Northern Virginia, was falling rapidly. ICPRB organized and coordinated efforts by both the National Weather Service and the U.S. Geological Survey to assess the risk of the reservoir falling to critical levels. As important, the demand reduction necessary to reduce that risk to an acceptable level was also determined. Two separate techniques for estimating risk and the impact of changing operating policies on risk were developed and implemented. These techniques are described in the next section.

Risk Analysis

Streamflow is a highly variable and unpredictable phenonmenon. Given a level of storage in a reservoir, there is always some risk that available storage plus future inflows will be sufficiently less than required withdrawals to cause the reservoir to fall to a critical level. The objective of risk analysis is to quantity that risk.

In its simplest form, risk analysis consists of a simulation of reservoir operation using historical streamflows. Starting the reservoir at its current level on the currrent date in any particular past year, and simulating the expected level of demand for water, one can determine the minimum level to which the reservoir would have fallen during that year. If all the years in the historical record are run through such a simulation, the number of historical years in which the reservoir would have dropped to critical levels can be counted. Dividing this by the total number of years in the historical record gives an estimate ot the probability of falling to the critical level. When this simulation was first performed on the Occoquan Reservoir on September 1, 1977, the risk was 15 percent (probability of .15). This risk was unacceptably high.

The technique just described does not take factors such as dryness of the soil, weather forecasts, or serial correlation of streamflows (i.e., once stream flows are low they tend to remain low) into account. Two different approaches were developed to remedy this situation. The first technique considered serial correlation, eliminating from the analysis those years in which streamflows in the previous three months were quite high. Because the previous months streamflows had been high, it was felt that the records from those years would not be representative of what might happen in the future. In addition, statistical techniques were used to extend the historical record of inflows to the reservoir by correlation with gauges on nearby streams.[12]

Serial correlation of streamflows in the Occoquan seems largely due to reductions in soil moisture, according to statistical analysis recently completed by the senior author. The second technique for risk analysis was based on this assumption. The National Weather Service's Extended Streamflow Prediction Model was calibrated to the Occoquan basin. This model simulated runoff from meteorological data. Soil moisture conditions in the model were set to estimated current conditions, and historical rainfall data, from the current date in each year, were then run through the model to produce a synthetic set of historical streamflow data conditioned by current soil moisture conditions. Since there seemed to be no serial correlation in the rainfall data, this entire synthetic record was then used in an analysis identical to the orignal streamflow analysis to determine risk.

An attempt was made to incorporate weather forecasts into this analysis. Those years in which upper air circulation patterns were most unlike the current patterns were eliminated from the analysis. This technique shows promise, although it produced little change in the results of the Occoquan analysis.

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[12] Risk Analyses for a Water-Supply System-Occoquan Reservoir, Fairfax and Prince William Counties, Virginia, Hirsch, R.M., U.S. Geological Survey, Open-File Report 78-452, 1978.

Both analyses showed that risk was quite sensitive to small reductions in demand, and that the risk level could be reduced almost five-fold by a 10 percent demand reduction. Achievement of this reduction was made the policy of the FCWA for the duration of the drought. The analyses were also used to determine the amount of storage necessary to keep risk at an acceptable level. This was given as a function of time and presented as the "Target Storage" in Figure 4.

Summary

This paper has attempted to describe the confused nature of Water Supply and Water Quality Management Planning in the Washington Metropolitan Area. This confusion is caused by large number of federal, state and local agencies which have decision making authority and/or regulatory authority in the area. ICPRB attempts to coordinate the actions of these agencies and to mediate disputes among them, and serves as a source of technical information. In performing this role, two new and useful planning and management tools have been developed. Both the Sewer Expansion Simulation and the Risk Analysis technique have been described. They are thought to have wide application, both in the United States and in other countries.

A Clean-Up Chronology

Washington Post 6/16/77

Compiled by the Interstate Commission on the Potomac River Basin

1608: Captain John Smith reported that the Potomac ". . . is fed with many small rivers and springs, frequented by otters, beavers, martins and sables. Neither better fish, more plenty, nor more variety for small fish had any of us ever seen in a place."

1700's: "The lower Potomac was alive with skipjacks and sloops carrying local commerce—tobacco, shad, herring and lumber."

1785: The Potomac River Fisheries Compact was signed. It guaranteed equitable fishing rights to Maryland and Virginia, but established Maryland as owner of the river.

1835: The Baltimore and Ohio Railroad was completed. Wastes from passengers discharged raw into the river at bridges.

1859: A public water supply was available in Washington for the first time.

1870: Eighty miles of sewers and culverts transferred untreated human wastes from Washington, D.C., into the Potomac.

1889: President Benjamin Harrison appointed an engineering board to plan a better sewage disposal system for Washington.

1894: A Public Health Service report stated that "at certain times of the year the river is so loaded with sediments as to be unfit for bathing as well as for drinking and cooking purposes. It contains fecal bacilli at all times, although these . . . are derived . . . from horses and cattle." This same year, flow records were started on the Potomac.

1900: During the first few years of this century, Washington, D.C.'s sewer outfalls were placed below the city. Water quality alongside the river improved temporarily.

1914: By this year, raw sewage from Washington's population of 350,000 was being dumped into the Potomac.

1930: Officials agreed that Washington's sewage should be treated before being discharged into the river.

1932: By this year, wastes from 575,000 people in the metropolitan area entered the river. Bacterial contamination closed the river to swimming from Three Sisters Island to Fort Washington.

1938: Blue Plains regional sewage treatment plant began operation. It was Washington's first treatment plant and provided only primary treatment, which is basically a settling of solids before effluent is released.

1948: The first permanent Federal Water Pollution Control Act was passed. It strengthened enforcement and research of water quality and initiated grants for construction of waste treatment facilities. It provided for establishing an enforcement conference that could set schedules for activities aimed at improving water quality. By this year, wastes from untreated and partially treated sewage of Washington's population of 1.6 million was double the 1932 load.

1959: Secondary treatment was added at Blue Plains. Secondary treatment provides aeration and/or filtration of sewage before effluent is released. Chemicals also may be used.

1965: The Water Quality Act was passed. It established a Federal Water Pollution Control Agency in the Department of Health, Education, and Welfare and required certain water quality standards.

1965: The Water Resources Planning Act established the Water Resources Council to plan for the development and protection of future water supplies.

1966: The Clean Water Restoration Act amended the Federal Water Pollution Control Act. It increased grants for research, state programs and construction of treatment plants.

1969: Chlorination began on the effluent of Washington area sewage treatment plants.

1969: The Potomac Enforcement Conference reconvened to examine the Potomac's water pollution problems. An official from the Federal Water Pollution Control Administration described the river as ". . . a severe threat to the health of anyone coming in contact with it because of bacterial contamination. . . The water was so low in dissolved oxygen in places that fish could not survive. The bottom of the Potomac was completely covered with an evil-smelling, thick, dark gray, sludge-like ooze. Repulsive debris, including solid materials from raw sewage, defiled the surface of the estuary in the vicinity of combined sewer overflows when it rains." That same year, the Army Corps of Engineers estimated the effects of agricultural runoff in the Potomac Basin as being equal to the untreated wastes of 3.5 million people.

1970: The Environmental Protection Agency was established, taking over water quality improvement programs.

1971: The D.C. Health Department prohibited water skiing, swimming and wading in the Potomac, Rock Creek and the Anacostia River. Officials decided chemical treatment would be used at Blue Plains until more sophisticated treatment procedures (advance wastewater treatment) were installed there.

1972: The Georgetown Gap was closed. The gap was a sewer and storm drainage outlet in Georgetown that released an overflow into the river estimated at between 15 and 25 million gallons of raw sewage per day.

1975: A study by the Interstate Commission on the Potomac River Basin showed no answers to whether water quality had improved. Below Washington, bacterial densities had decreased since the late '60s due to chlorination of sewage effluents, but problems remained with algae growths, suspended solids and decreasing oxygen for plant life.

1976: Arsenic was discovered on the Potomac shoreline in Alexandria at the site of a chemical plant. The Environmental Protection Agency rejected a plan for constructing the Dickerson sewage treatment plant, which would have been on the Maryland shoreline of the river above major water supply withdrawals. Blue Plains reduced sludge production, indicating that there was a temporary reduction in levels of treatment to sewage effluent before it was discharged into the river. A bass was caught in the Washington Channel.

1977: Blue Plains improved secondary treatment facilities, marking a reduction in the volume of pollutants entering the Potomac's upper estuary.

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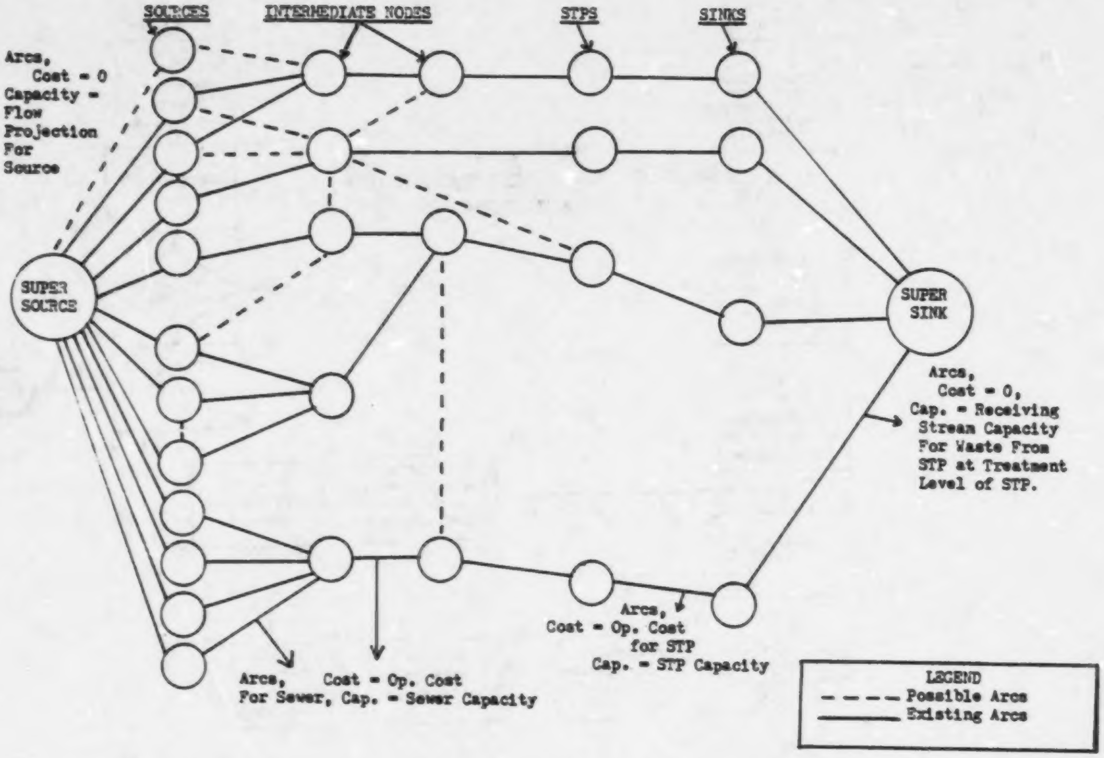
FIGURE 1

NETWORK FEATURES


SOURCES
INTERMEDIATE NODES
STPS
SINKS
Arcs, Cost = 0 Capacity = Flow Projection For Source
SUPER SOURCE
SUPER SINK
Arcs, Cost = 0, Cap. = Receiving Stream Capacity For Waste From STP at Treatment Level of STP.
Arcs, Cost = Op. Cost for STP Cap. = STP Capacity
Arcs, Cost = Op. Cost For Sewer, Cap. = Sewer Capacity
LEGEND
Possible Arcs
Existing Arcs

1. Supersource - Artificial node, origin of all flows.
2. Arcs Between the Supersource and Sources - Operating costs on these arcs is 0 (0 cost = 0). Capacities on the arcs are set equal to the projected flow from the appropriate source for the appropriate year. The arcs channel the circulation through the appropriate sources.
3. Sources - Contiguous areas directly tributary to any STP or conveyance facility which is to be considered.
4. Arcs Between Sources or STPs and Intermediate Nodes or Between Intermediate Nodes - These are unidirectional arcs with capacities equal to the capacity of the conveyance represented and operating costs proportional to the flow in the OKA, they are assumed constants for existing facilities and added after optimization.
5. Intermediate Nodes - Intermediate nodes will be located anywhere a junction between existing facilities and possible future facilities may occur. Intermediate nodes also exist at existing junctions and any possible junctions between non-existant facilities to be considered in any alternatives.
6. STPs - STPs.
7. Arcs Between STPs and Sinks - Capacities on these arcs are the hydraulic capacities of the plants. Costs on the arcs are operating costs per MGD. Maintainence costs are considered as in 4 above.
8. Sinks - Receiving streams.
9. Arcs Between Sinks and Supersink - The operating cost on these arcs is 0. The capacities on the arcs are equal to the allowable loadings from the STPs in MGD of effluent. Capacities on these arcs are altered by improving treatment levels at the STPs.
10. Supersink - Artificial node, sink for all flows.

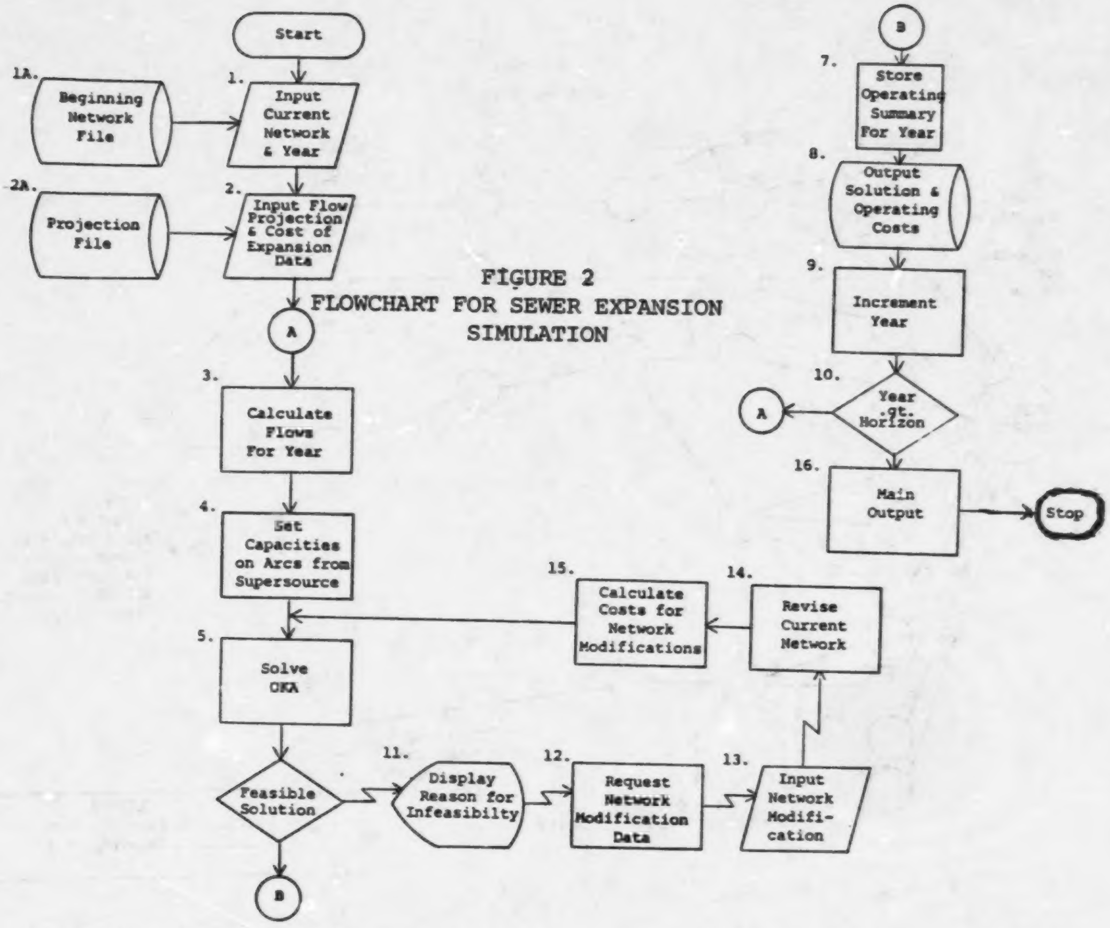

Start
1A.
Beginning Network File
1.
Input Current Network & Year
2A.
Projection File
2.
Input Flow Projection & Cost of Expansion Data
A
3.
Calculate Flows For Year
4.
Set Capacities on Arcs from Supersource
5.
Solve OKA
Feasible Solution
B
11.
Display Reason for Infeasibilty
12.
Request Network Modification Data
13.
Input Network Modifi-cation
14.
Revise Current Network
15.
Calculate Costs for Network Modifications
B
7.
Store Operating Summary For Year
8.
Output Solution & Operating Costs
9.
Increment Year
10.
Year .gt. Horizon
A
16.
Main Output
Stop

FIGURE 2
FLOWCHART FOR SEWER EXPANSION SIMULATION

1)Input Current Network and Year - from beginning network file. This is a simple read of the file 1A into appropriate core storage metrices for use by the OKA in step 7.

1A)Beginning Network File - This is a disc file on which resides the following information, a) identification of nodes as sources, sinks, or intermediate nodes; b) upper and lower bounds on flows on existing arcs in the network; c) operating cost functions for all existing arcs; and d) a starting solution for the OKA on the network. Also includes starting year.

2) Input Flow Projection and Cost of Expansion Data - from projection file. This is also a simple read from the file into appropriate core storage for use when updating the network or solving the OKA on a revised network.

2A) Projection File - The file contains flow projections for each source in the network, costs for reducing flows at those sources (if appropriate), and costs for modifying any arc in the network (e.g., expanding or constructing a sewage treatment plant, or abandoning such a plant, or expanding or constructing new conveyance facilities). Costs for upgrading treatment on arcs from the sinks to the supersink at STPs must also be included.

3)Calculate Flows for Year - Uses variable "year" and data from projections file.

4)Set Capacities on Arcs from Supersource - equal to flow projections for respective sources. Uses results of step 3 and insures OKA circulation is channeled through the network in a realistic fashion.

5) Solve OKA - on current network for circulation equal to sum of flow projections. Efficiently allocates projected flows for year to available treatment facilities through available interceptors.

6) Feasible Solution - Branching operation on feasibility of solution from step 5.

7 Store Operating Summary for Year - Appropriate data from step 5 solution is stored for use in step 16.

8 Output Solution and Operating Costs - Output detailed operating information from step 5 solution.

9 Increment Year - by one year.

10 Year .gt. Horizon - Branch for end of simulation.

11 Display Reasons for Infeasibility - Calculates reasons for infeasibility from step 5 infeasible solution variables and dual variables and displays information on interactive terminal to assist user in selecting appropriate network modifications.

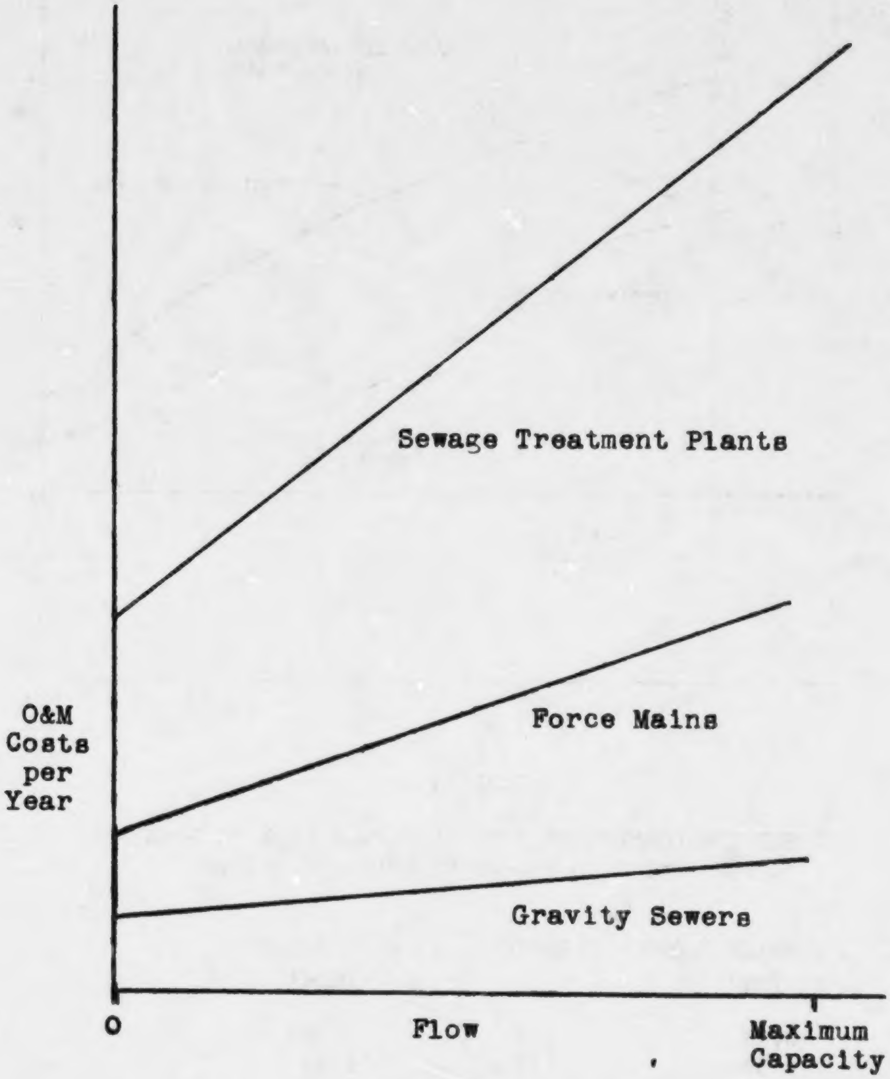


Figure 3 Idealized Cost Curves for Operation and Maintainence

FIGURE 4

OCCOQUAN TARGET STORAGE CURVE

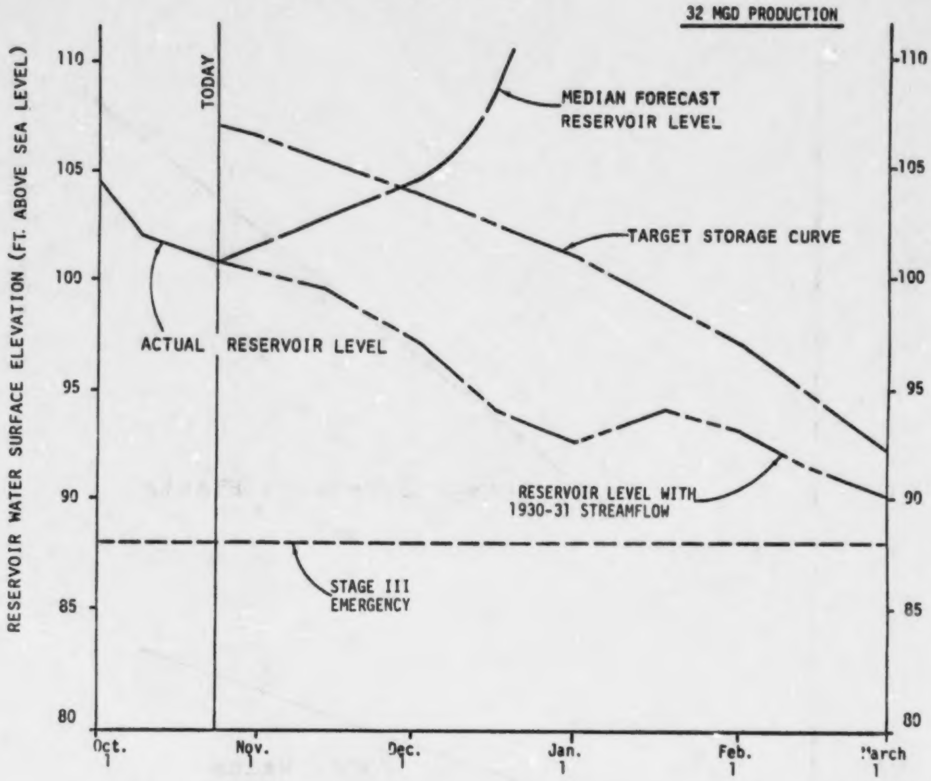


TABLE 1

TABLE FOR CONVERTING AVERAGE SEWAGE FLOW TO FLOW FOR DESIGN IN SEWERS OF SEPARATE SYSTEM

AVERAGE FLOWS (mgd)	RATIO	PEAK FLOWS (mgd)
0.10	1:4	0.40
0.50	1:3.6	1.81
1.00	1:3.2	3.18
2.00	1:2.8	5.62
5.00	1:2.4	11.94
10.00	1:2.1	21.12
14.00	1:2.1	28.76
18.50	1:2	37.00

*For flows above 18.50 mgd use ratio of 1:2

Appendix - Interactive Output From SES

```
TIME-09:48:40 CPU-00:00:00 SERVICE-679 SESSION-00:01:11 OCTOBER 19, 1978

MONTGOMERY COUNTY CALIBRATED DATA ALL PLANTS OPERATING 1978 WSSC DATA 1978
SKIP TO NEXT DATA DECK?  (YES, NO, QUIT)
no

PROJECTIONS UPDATED TO 1978
NO OF ARCS = 268 NO OF NODES = 143

NO FEASIBLE FLOWS FOUND FOR 1978
THE FOLLOWING NODES CONTRIBUTE TO PROBLEM:  (ARCS LISTED ARE SITES OF
OVERFLOWS)
7A0301
     7A0301  7A0401  1.199
THESE SEWERSHED ARCS ARE CONSTRAINED BY LOWER BOUNDS:
     SSRCE   7A0301  1.313

DO YOU WISH TO WRITE A FILE?  (YES, NO, SYSTEM, QUIT)
'SYSTEM' WILL LIST ALL INFORMATION AT THE TERMINAL-
no

IF YOU NEED STATISTICS TYPE "INFO", ELSE TYPE "NO"
no

ENTER DESIRED ALTERATION:
BEGINNING NODE?
7a0301

ENDING NODE?
7a0401

FACILITY TYPE?  (STP, LTF, SWR, FMN)
swr

NEW CAPACITY ON ARC?
?
1.2

NEW FACILITY?  (IF OLD, TYPE "ABD" OR "UPG")
new

LENGTH (THOUSANDS OF FEET)?
?
4.54
```

```
COMPUTED COST:  CAP, FOM, VOM:  0.315  0.0  0.0
TO IMPLEMENT, TYPE "YES":  TO INPUT DIFFERENT COSTS, TYPE "NO"
yes

ANY OTHER ALTERATIONS?  (YES, NO, QUIT)
no

FEASIBLE FLOW FOUND FOR 1978
             .
             .
             .
             .
             .

FEASIBLE FLOW FOUND FOR 1995

DO YOU WISH TO WRITE A FILE?  (YES, NO, SYSTEM, QUIT)
'SYSTEM' WILL LIST ALL INFORMATION AT THE TERMINAL-
no

NO MORE DATA IN PROJECTIONS FILE
"QUIT" HAS BEEN CALLED.  DO YOU WISH FINAL OUTPUT?
yes

              TYPE CAPACITY  O&M/THIS/PERIOD   CAPITAL   BOD       FLOW
SOURCE  SINK   FAC  CHANGE  (FIXED)  (VAR.)     COSTS  CONCENT.  7-Q-10/CFS
1978
7A0301  7A0401  3   1.200    0.0     0.0        0.315
SUMMARY OF COSTS FOR 1978
CAPITAL   0.315 O&M FIXED    0.0     VARIABLE   0.0
1985
7C0101  7M0301  4   1.000    0.020   0.059      0.984
7E0601  SSINK   1   2.000    0.039   0.226      5.291   8.000     10.000
7G1401  7G1301  3   2.000    0.0     0.0        0.315
7E0101  7B0701  4   2.000    0.021   0.031      1.050
7G0301  7G0501  3   3.000    0.0     0.0        0.315
SUMMARY OF COSTS FOR 1985
CAPITAL   7.955 O&M.FIXED    0.077   VARIABLE   0.172
1995
7A0801  8A9999  3   4.000    0.0     0.0        0.0
SUMMARY OF COSTS FOR 1995
CAPITAL   0.0   O&M FIXED    0.077   VARIABLE   0.208
1995 IS THE LAST YEAR COMPLETED.  ONLY COSTS THROUGH 1995
ARE INCLUDED IN THE PRESENT VALUE ANALYSIS
BASE YEAR FOR THE PRESENT VALUE ANALYSIS IS 1978
P.V. CAPITAL COSTS           6.493
P.V. FIXED O&M COSTS         0.570
P.V. VARIABLE O&M COSTS      1.354
TOTAL PRESENT VALUE THROUGH
1995                         8.417
TIME-10:05:17 CPU-00:00:04 SERVICE-8442 SESSION-00:17:47
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THE OCCOQUAN WATERSHED POLICY -

A COMPREHENSIVE PROGRAM TO SAVE A WATER SUPPLY

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THE OCCOQUAN WATERSHED POLICY - A COMPREHENSIVE PROGRAM TO SAVE A WATER SUPPLY

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ABSTRACT

The Occoquan Reservoir currently serves as the water supply for 600,000 people in Northern Virginia. The preservation of this supply in the water-poor Washington metropolitan area has required considerable effort by the State and local governments. Initial efforts involved attempts to control growth by limits on the size of proposed sewage treatment plants. This was recognized as only a stop-gap measure and met with limited success. Following extensive studies and public hearings, the Virginia State Water Control Board in July 1971, adopted a "Policy for Waste Treatment and Water Quality Management in the Occoquan Watershed." A principal element of this Policy was the requirement for a highly sophisticated regional advanced waste treatment plant to replace 11 existing sewage treatment plants in the Occoquan Watershed. The Upper Occoquan Sewage Authority (UOSA) was created to fulfill this requirement of the Policy.

The construction of the UOSA plant was delayed by limited available federal construction grant funds and changing state and federal rules and regulations. The plant has now been operational four months and the plant performance has been very good. The long range impact of the plant on the Occoquan Reservoir will be demonstrated by the monitoring data collected in the future.

HISTORY

The Occoquan Watershed is located southwest of Washington, D. C., a short distance beyond the highly developed urban area surrounding the City (Figure 1). The Watershed encompasses 600 square miles, 570 of which drain to the Occoquan Reservoir. The Reservoir serves as the water supply source for 600,000 people in Alexandria, Fairfax County, and portions of Prince William County.

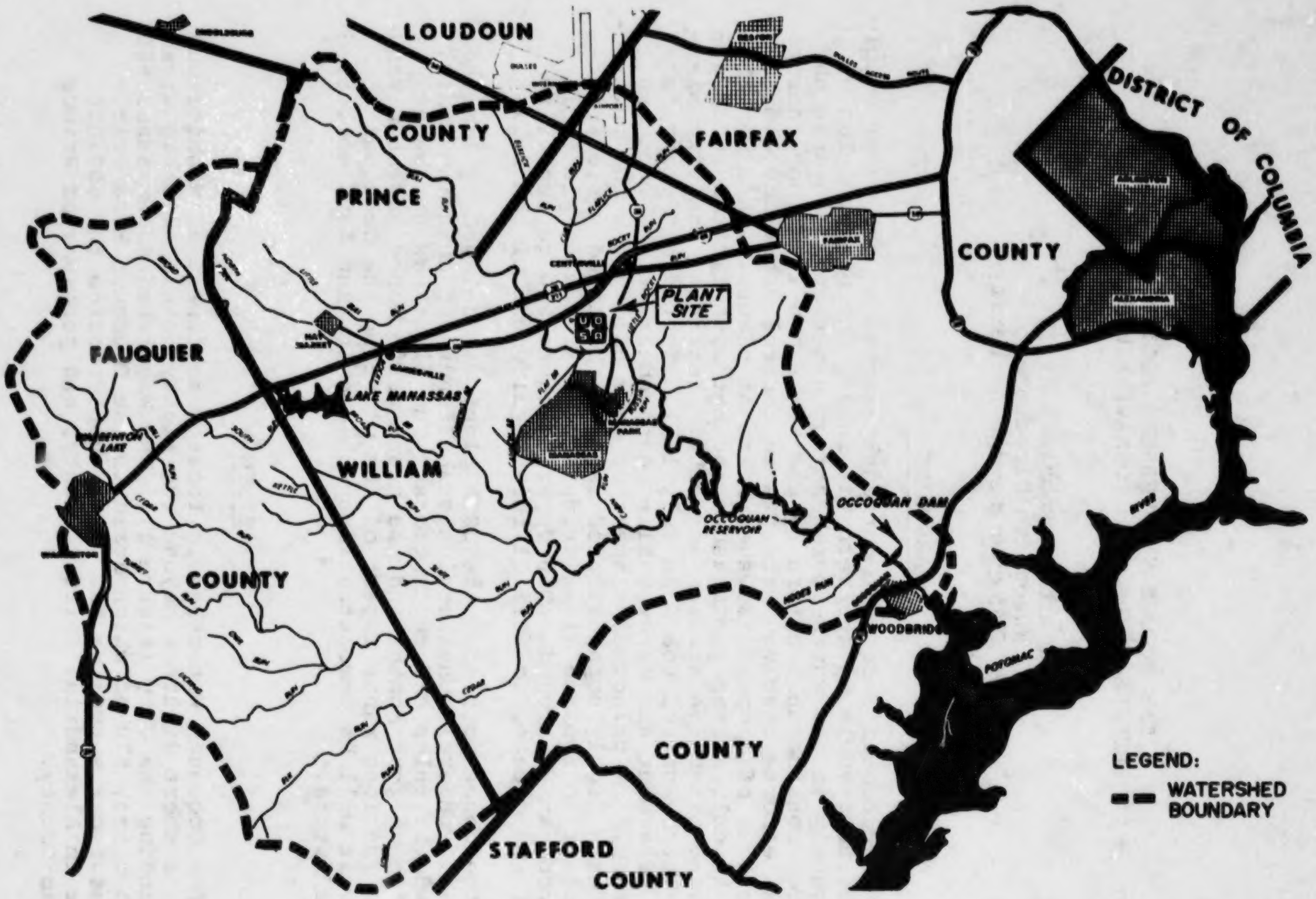

FIGURE I- GENERAL LOCATION OF OCCOQUAN WATERSHED AND UOSA PROJECT

The first impoundment in the watershed was created by a low head dam constructed near Occoquan, Virginia in 1950. This impoundment served to supplement a number of small supplies until 1957 when a new, or upper dam was constructed to meet the needs of a rapidly increasing population. This population growth is reflected in Table 1.

Despite the rapid population increase in the Washington metropolitan area, the Occoquan Watershed until the 1960's remained largely rural. However, with the construction of the interstate highway system, the Watershed became within easy commuting distance of Washington. This afforded the persons employed in the Nation's Capital an opportunity to live in a more rural and less congested area than available in Washington and its immediate surroundings. The housing also was available at much lower costs than similar housing in Washington. Consequently, pressure to expand and develop new housing in the watershed increased tremendously.

TABLE 1. AREA POPULATION

Jurisdiction	1950[(a)]	1960[(a)]	1970[(a)]	1975[(b)]
Fairfax County	98,557	275,002	407,901	537,200
Prince William Co.	22,612	59,061	88,933	136,600
Alexandria	61,787	91,023	110,938	113,900
Totals	182,956	425,086	607,772	787,700

Source:

(a) Census Bureau of the United States.
(b) Planning Office of each jurisdiction.

The Virginia State Water Control Board recognized that the projected population expansion and continued use of conventional wastewater treatment methods presented a serious threat to the water supply. Initially, the Board's efforts to protect the water supply involved limiting the number and size of sewage treatment plants in the Watershed. This policy served to slow the deterioration of the Reservoir, but had a severe adverse impact on the commercial and industrial well-being of the area. The policy also resulted in many small, inefficient publicly and privately owned secondary sewage treatment plants with highly variable performance.

Recognizing that the long-term solution to the problem required a coordinated, basin-wide comprehensive plan, the Board in 1969 employed Metcalf & Eddy, consulting engineers, to study the Watershed and recommend a program for preservation of the Reservoir as a water supply.

The consultants presented their results and conclusions in the Spring and Summer of 1970.(1)(2) Their principal findings were:

1. The Reservoir was highly eutrophic in the Summer and early Fall. During this period, the Reservoir had heavy algal blooms, mainly blue-green algae, and the water below 10 feet in depth was anaerobic.

2. If the algal nutrient load increased, the conditions in the Reservoir would continue to deteriorate as the algal blooms progressed down the reservoir toward the water supply intake.

3. The sewage treatment plant effluents were mainly responsible for the advanced stage of eutrophication occurring in the Occoquan Reservoir and this condition would become progressively worse as sewered populations increased.

4. The nutrients contained in natural drainage from forested, agricultural, and urban lands, were sufficient to support algal blooms that may be occasionally troublesome. However, the frequency and intensity of the blooms would be greatly reduced in the absence of treated sewage nutrients and the type of the blooms may change in character, containing much less of the obnoxious blue-green forms.

Metcalf & Eddy offered the following three alternate solutions to the problem, listed in order of the consultants preference.

1. Export of all wastewater out of the Occoquan Watershed.

2. Provide the most advanced methods of wastewater treatment, pipe the effluent to the Fairfax County Water Authority for direct reuse, and limit the population to produce the saleable amount of water.

3. Provide the most advanced methods of wastewater treatment and limit the sewered population to 100,000 people or a total treated wastewater capacity of 10 million gallons per day (MGD).

Following review of the reports by the affected localities and state agencies, the State Water Control Board held a series of hearings to receive testimony from the general public. This planning effort culminated in July, 1971, with the adoption by the State Water Control Board of "A Policy for Waste Treatment and Water Quality Management in the Occoquan Watershed", commonly referred to as the Occoquan Policy.(3) This Policy is a document of considerable scope and has been cited as a model comprehensive water quality plan as envisioned in Section 208 of Public Law 92-500, which legislation was adopted over 3 years after the beginning of the effort to develop the Occoquan Policy.

The State Water Control Board in the Occoquan Policy chose alternative 3, e.g., the most advanced waste treatment and limiting of population as the most cost effective method to protect the watershed.

Principal reasons for rejection of the other 2 alternatives were:

1. Alternative 1, the export of all wastewater, was too expensive and resulted in the transfer of a potential pollution problem to another watershed. The initial capital cost of the export plan was estimated to be $260 million with a total cost by 1990 of $442 million.

2. Alternative 2, the direct piping of the effluent to the water authority, was unacceptable to the State Health Department.

The Occoquan Policy mandated the following actions:

1. A regional water reclamation plant utilizing the best treatment technology available would replace the existing sewage treatment plants in the watershed. The effluent requirements for the regional plant were the most stringent in the United States (Table 2).

2. Only high performance, treatment techniques proven on a plant scale would be utilized in the regional plant. Unproven technologies could be utilized only to meet treatment standards for which demonstrated techniques were not available.

3. The regional system reliability would be enhanced by the following facilities at the plant and all pump stations.

   (a) Stand-by units for all electrical and mechanical components;

   (b) Three separate sources of electric power; and

   (c) Emergency holding basins.

4. Until the regional plant was available, all existing

TABLE 2. MINIMUM EFFLUENT QUALITY REQUIREMENTS* FOR A PLANT ON THE OCCOQUAN WATERSHED

	BOD mg/1	COD mg/1	SUSPENDED SOLIDS mg/1	NITROGEN mg/1	PHOSPHORUS mg/1	MBAS mg/1	TURBIDITY Jax Units	COLIFORM PER 100 ml SAMPLE
Final Effluent Requirements	1.0	10.0	<1.0	1**	0.1	0.1	0.4	Less than 2

*As measured on a weekly average..

**When technology is available this requirement must be met with a reliable year-round nitrogen removal capability. Accordingly, in the detailed design of the initial plant, the designer should consider incorporating chemical means to achieve this lower limit, if EPA research work develops a proven, reliable all-weather unit. Until that time, the plant shall be designed such that its effluent contains not greater than 1 mg/1 of unoxidized nitrogen.

sewage treatment plants would use chemical additions to increase nutrient removals.

5. A sub-committee was required to monitor the condition of the Reservoir and its tributaries.

6. The initial capacity of the regional plant would be limited to 10 MGD and the plant could be expanded in 5 MGD increments provided the plant met the effluent discharge requirements and water quality in the Reservoir remained satisfactory.

THE UPPER OCCOQUAN SEWAGE AUTHORITY

The local governments created the Upper Occoquan Sewage Authority (UOSA) to construct and operate the regional advanced waste treatment plant. The first major action of the Authority was the employment of CH_2M Hill as consulting engineers on the project.

The project design and development of the financial program proceeded simultaneously and although the multijurisdictional service contract with UOSA produced extensive debate, these two phases of the project were accomplished on schedule.

The final plans and specifications were submitted to the State for review in May, 1973, andthe project schedule provided for award of the treatment plant contract in September, 1973, with completion in 26 months. However, because of an unanticipated limit on federal grant funds, UOSA was advised that the project had to be split into several contracts to be awarded as federal grant funds became available. This project splitting delayed the completion of the plant by three years and increased the cost by an estimated $33 million.(4) The total project cost is now projected to be $82 million including the necessary interceptor sewers and pump stations to deliver the flows from the 11 existing plants.

DESCRIPTION OF THE UOSA PLANT

A schematic of the UOSA treatment plant is presented in Figure 2. The first treatment processes are fairly conventional primary and secondary treatment. A unique feature of this plant section is the capability to divert a portion or all of the plant influent or primary effluent to an emergency retention pond with a capacity of 45 million gallons. The contents of this pond are returned to the plant headworks after the "emergency".

A computer is used to monitor most of the critical

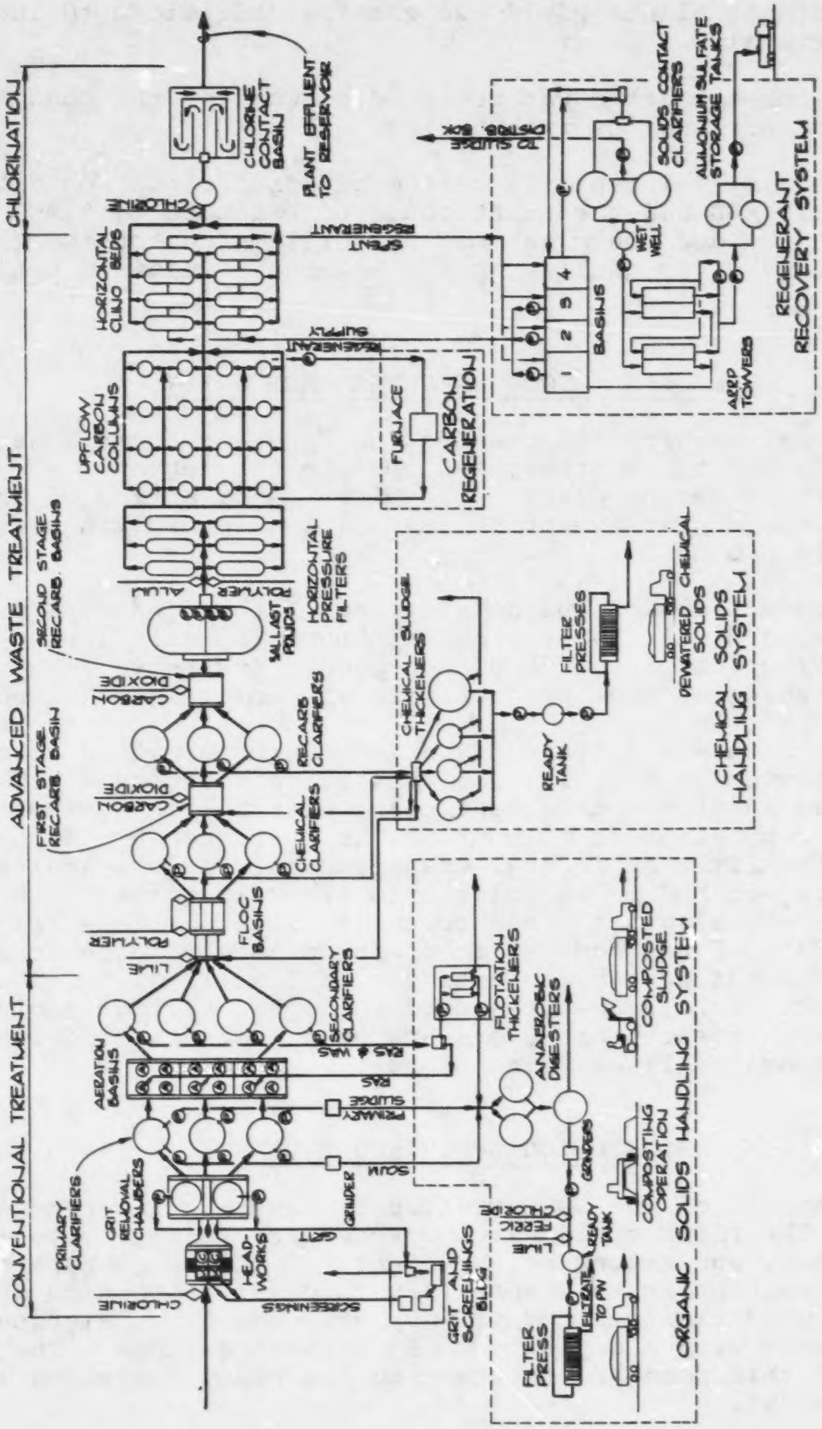

FIGURE 2-PLANT SCHEMATIC FLOW DIAGRAM

functions in the UOSA system and the secondary portion of the plant involves the first use of the plant computer for process control. The dissolved oxygen in the completely mixed activated sludge process is controlled by varying the depth of submergence of the aerator impellers. The impeller submergence is varied by computer adjustment of the aeration basin discharge weirs based on the dissolved oxygen levels.

The advanced waste treatment portion of the plant begins with chemical clarification utilizing lime to precipitate phosphorus. Lime additions are computer controlled to raise the pH to 11.3. The precipitate is then flocculated and settled. The pH is reduced to 7.0 by two-stage recarbonation with intermediate settling.

Ballast ponds or equalization basins are used to provide constant flows to the remaining processes.

The next treatment process is pressure filtration to remove the remaining particulate matter. The filtering action is enhanced by employing three filter media of different specific gravity. The largest media has the lowest specific gravity and the smallest media the highest specific gravity, thus promoting a decreasing media size through the bed.

Following filtration, the water passes through carbon adsorption columns, ion exchange columns for ammonia removal, chlorination and discharge to a final effluent reservoir with several days detention. The plant performance parameters are continuously measured in the discharge from the chlorine contack tank, and if a parameter exceeds its limit, the computer closes the discharge valve and the unacceptable water is recycled for additional treatment.

The organic solids are processed by two-stage anaerobic digestion, dewatering on a filter press, and composting. The compost will be used by local park authorities as a soil amendment in their nurseries. The methane from anaerobic digestion is used to fire the boilers to heat the digesters and produce steam for plant heating, and the stack gases are used in recarbonation.

The chemical solids are first gravity thickened and then dewatered on filter presses. The chemical solids are presently landfilled, but UOSA in cooperation with the Virginia Department of Agriculture, plans to explore agricultural uses for the chemical solids.

The ion exchange beds are regenerated by purging the ammonia with a concentrated sodium chloride regenerant solution and the ammonia is removed from the regenerant solution by the Ammonia Recovery and Regeneration Process (ARRP). The ARRP process consists of a closed air stripping and absorption system. Initially, the regenerant solution pH is raised to volatilize the ammonia. The ammonia gas next is stripped into an air stream in a stripping tower and the air stream containing the ammonia then flows through an absorption tower. The absorbent liquid is sulfuric acid and the ammonium ions are absorbed as ammonium sulfate. The ammonium sulfate will be marketed as a fertilizer.

The initial approved capacity of the UOSA plant is 10.9 MGD. After successful performance for one year, the plant permit capacity can be increased to 15.0 MGD. The plant permit capacity of course, does not include the capacity of the standby units required by the Occoquan Policy.

PLANT OPERATIONS

The UOSA plant began operations on June 26 of this year. Plant construction is not completed, and several facilities are not available for use by the Authority. However, all unit processes are operative except the ion exchange facilities for ammonia removal.

The performance to date has been very good. By utilization of the plant's extensive holding facilities, we were able to recycle the first few days production for additional treatment, and as a result, the first effluent discharged from the plant on July 10 met most of the permit limits.

A summary of the performance data for September, 1978, is presented in Table 3.

OPERATION AND MAINTENANCE COST

Based on an initial flow of 8.0 MGD, the operations and maintenance cost is projected to be 90¢ per 1,000 gallons. The operational experience to date indicates that this is a reasonable projection.

In Table 4, the UOSA operation and maintenance cost is compared to the costs incurred by some of the jurisdictions to operate their small secondary treatment plants replaced by UOSA. This comparison shows that due to economy of scale and certain efficiencies incorporated in the UOSA design, the UOSA plant is able to produce a very high quality water for a cost comparable to the cost to the jurisdictions to operate their own small secondary treatment plants.

TABLE 3. PERFORMANCE REPORT - SEPTEMBER 1978

Parameter	Permit Limit	Units	PLANT PERFORMANCE - WEEKLY AVERAGES			
			1st Week	2nd Week	3rd Week	4th Week
BOD	1.0	MG/1	0.96	1.05	0.87	1.00
Suspended Solids	<1.0	MG/1	0.84	0.44	0.69	0.52
Total Phosphorus	0.1	MG/1	0.04	0.02	0.02	0.08
*TKN	1.0	MG/1	2.91	7.57	8.41	1.14
COD	10.0	MG/1	2.70	2.36	1.96	1.90
MBAS	0.1	MG/1	0.1	-	-	0.1
Turbidity	0.4	NTU	0.38	0.35	0.40	0.43
Fecal Coliform	2.0	Number/ 100 ml	<2.0	<2.0	<2.0	<2.0

*TKN - This permit limit is not applicable until nitrogen removal facilities are completed.

TABLE 4. COMPARISON OF OPERATION AND MAINTENANCE COSTS

Jurisdiction	Number Plants	Operation and Maintenance Cost per 1,000 Gallons
Fairfax County	5	$1.47
Prince William Co.	1	.43
City of Manassas	2	.71
UOSA		$ .90

CONCLUSIONS

The impact of the UOSA plant on the water quality in the Occoquan Reservoir can be determined from the data of the Occoquan Watershed Monitoring Laboratory. This Laboratory, also required by the Occoquan Policy, is operated by Virginia Polytechnic Institute and State University. The Laboratory reports that the water quality in the Bull Run arm of the Reservoir this fall is the best ever observed in the six years of the monitoring program. This is a dramatic change from the extremely poor water quality observed in the Reservoir last fall.(5)

The full impact of the UOSA plant will not be known for sometime. Substantial additional development is planned for the watershed and if this development is not carefully planned and controlled, the pollutants in storm water run-off could cause a deterioration in Reservoir water quality. However, the preliminary evidence indicates that the UOSA plant has eliminated the water quality problems in the Reservoir caused by sewage effluents.

References

1. *A Comprehensive Pollution Abatement Program for the Occoquan Watershed* an Interim Report to the Commonwealth of Virginia, State Water Control Board, by Metcalf & Eddy, Engineers, March 18, 1970.

2. *1969 Occoquan Reservoir Study*, by Metcalf & Eddy, Engineers, to Commonwealth of Virginia, State Water Control Board, dated April, 1970.

3. *Policy for Waste Treatment and Water Quality Management in the Occoquan Watershed* adopted by the Virginia State Water Control Board on July 26, 1971.

4. *Summary Report Concerning EPA's "Operable Unit" Requirement and its Effects on the Capital Cost of the UOSA Project* a Position Paper adopted by the Upper Occoquan Sewage Authority Board of Directors on October 19, 1978.

5. *The Occoquan Quarterly - Summer 1977 Data*, a report issued by the Occoquan Watershed Monitoring Laboratory.

OPERATIONAL EXPERIENCE - SURFACT

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Introduction

This paper describes operational experience with a new treatment process termed SURFACT. Eleven months of operating experience and data are summarized and conclusions developed. Based on the information developed SURFACT is a viable secondary treatment alternate for either upgrading or new construction purposes.

Background

Philadelphia operates three wastewater treatment plants which serve a population in excess of 2 million and a drainage area of 130 square miles. The three plants, Northeast, Southeast and Southwest are collectively treating over 500 million gallons daily. Northeast is an intermediate activated sludge plant while the two Southern plants provide primary treatment.

Philadelphia is in the midst of a 600 million dollar expansion program aimed at raising the level of treatment and increasing hydraulic capacity to 600 million gallons per day. All three plants will be full secondary with BOD removal efficiencies equal to or exceeding 90 percent.

Over the last five years Philadelphia has conducted numerous pilot plant programs for its proposed secondary treatment facilities. Pilot work included oxygen activated sludge, physical chemical treatment, air activated sludge and rotating biological contactors. Late in the piloting program the Department felt that by merging air activated sludge with rotating biological contactors advantages of both systems would be available.

Since Autotrol's BIOSURF was used for pilot work and the process merged BIOSURF's with activated sludge, the name SURFACT was adopted.

Process Description

SURFACT physically merges a rotating biological contactor (RBC) with a diffused air activated sludge tank. Figure 1 shows how the two processes are

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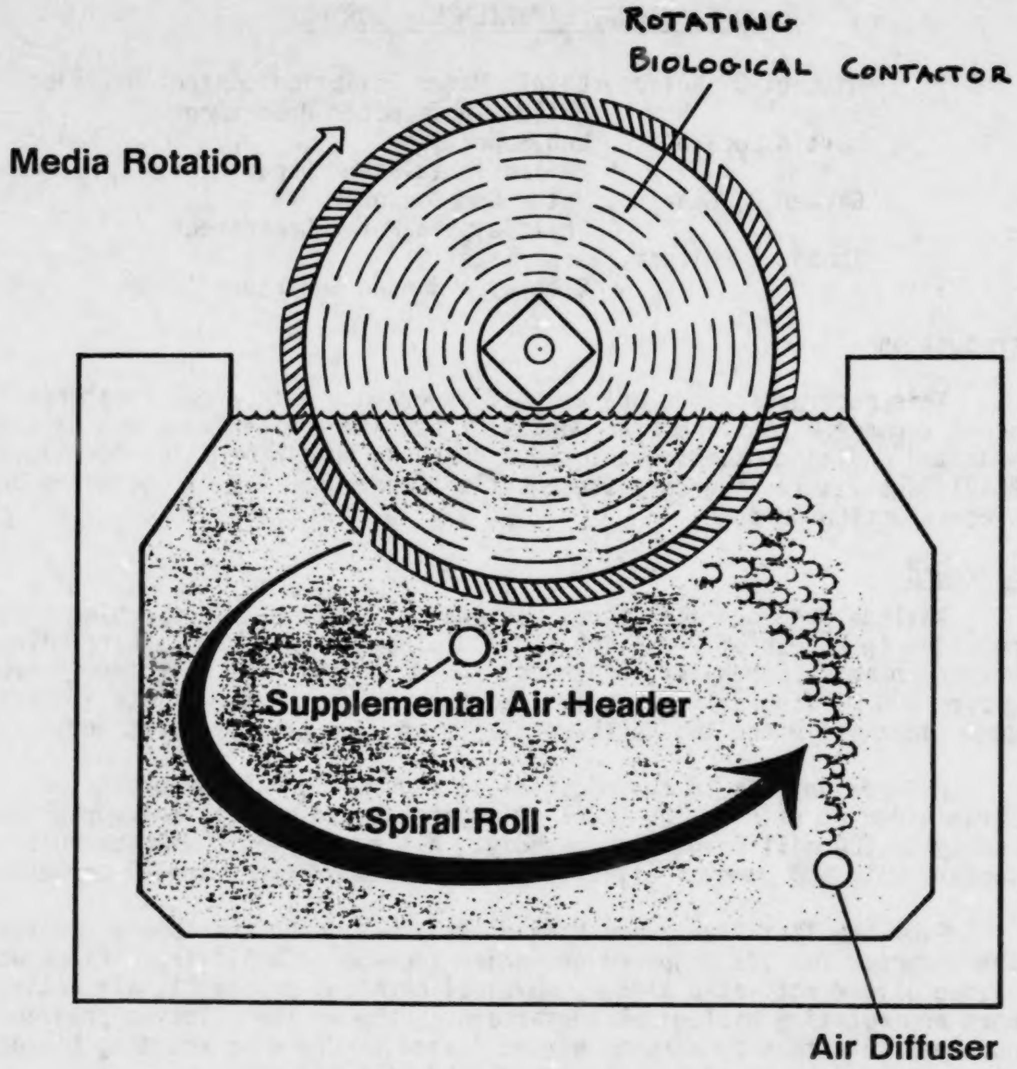
ROTATING BIOLOGICAL CONTACTOR
Media Rotation
Supplemental Air Header
Spiral Roll
Air Diffuser

FIGURE 1

merged.

In a diffused air aeration tank the rotating biological contactor is submerged to about 40% in the mixed liquor and spinning at approximately 1½ rpm. The contactor obtains its rotation via air, mechanically or by harnessing the hydraulic motion of a spiral roll tank. Biological growth on the contactor ranges from 40,000-70,000 mg/l suspended solids while the mixed liquor in the tank will be in the 3000 mg/l range. The rotating media are 11'5" in diameter constructed of polyethylene and provide 4000 sq. ft. of surface area/lineal foot. If the growth on the rotating media and mixed liquor were combined, an equivalent aerator concentration of 5000 mg/l would result.

Advantages

The following advantages for SURFACT were predicted:

1) greater biomass without an additional oxygenation load or final tank solids loading,

2) rapid upgrading of diffused air plants due to nominal construction and low hydraulic head loss for the rotating media,

3) lower energy costs,

4) lower capital costs for upgrading or new plant construction,

5) simple operation and maintenance,

6) potential for one tank nitrification

The first advantage results from carrying a significant portion of the biomass on the rotating media. Disc growth is self supporting from the oxygenation standpoint and does not, with the exception of minimal sloughing, require settling and recycling. Therefore a lower system F/M may be carried without adding more oxygen or clarifiers.

Advantage number two speaks to the straightforward construction aspects of upgrading. It was envisioned that the discs could be placed in aerator tankage as quickly as supports could be constructed.

Energy consumption for the merged system would be low. Little energy is required to drive the discs, particularly if the spiral roll of a tank is harnessed and no energy is required for oxygenation above that provided for the original mixed liquor.

In the case of capital costs the department judged that construction costs for upgrading would be low since better treatment would be possible without increases to aerator or final tankage. For new plant construction, since the potential for smaller aerators and final tankage exists, lower costs seemed likely.

Simplicity of operation and maintenance were felt to be another possible advantage. This was based on our previous pilot experience with rotating biological contactors.

Lastly, nitrifying in one tank and thereby eliminating intermediate settling and second stage aeration seemed possible under the proper loading conditions.

Preliminary Pilot Operations

Based on the possible advantages of this system, pilot work was initiated in the Fall '74. A four-foot long by 11'5" diameter rotating biological contactor with a surface area of 16,000 ft^2 was utilized for the pilot program. All testing was performed at our Northeast Water Pollution Control Plant in one of the aerators used for modified aeration. The primary object of these studies was to determine whether the RBC units would rotate by placing them in the aeration tanks. Other pilot objectives included observations for structural or clogging problems of the media. The studies also included observation of rotating biological contactors (RBC) rotational speeds during which air was applied through a special diffuser pipe directly under the RBC unit installed in accordance with the manufacturer's recommendations. The unit was tested at two tank locations and observations of the biological growth were made at each location. Preliminary results were favorable.

The resulting microbiology on the RBC unit was further advanced and highly active when compared to the mixed liquor solids. Although the primary objective of this phase of the program was to test the mechanical ability of an air drive RBC system, these micro biological results were promising.

Additional testing of the air drive system continued with the installation of a full scale RBC unit (25' length x 11'5" diameter) in the same tank. This pilot effort results in the configuration of the RBC's cups such that the media rotates with the surface hydraulic motion. On this basis the maximum existing energy is captured minimizing power requirements.

Prototype Installation

Based on the results of preliminary pilot operations, the department constructed a prototype installation for the Northeast plant. Operating criteria for the existing Northeast facility include:

Flow	190 MGD
Primary Tankage	
Detention time	1.0 hours
SOR	1800 gal/day/ft^2
Aerator Tankage	
Detention time	2 hours
Air rate	0.5 cf/gal
Final Tankage	
Detention time	2 hours
SOR	1050 gal/day/ft^2

Northeast can utilize modified aeration, step aeration and contact stabilization in various combinations. For the last several years the modified process has been employed. Mixed liquor levels have averaged 1300 mg/l with overall plant removals of BOD and suspended solids in the 60-75% range. An overall plant schematic is shown in Figure 2. Since aerator tankage is of the spiral roll diffused air variety, a prototype was installed simply by retro fitting two bays with rotating biological contactors. Final tanks serving the SURFACT bays were isolated so return sludges would not be co-mingled. Such an installation would illustrate the performance characteristics of the SURFACT process on a large scale basis and also provide comparison with adjoining tanks being operated in the modified aeration mode. Prototype Design Criteria follow:

Design Flow, MGD	20
Aeration Tank	
RBC Shafts, number	22
Shaft Length, ft	25
Media Diameter, ft	12
Area per Shaft, ft^2	100,000
Total RBC Shaft, $10^6 ft^2$	2.2
RBC Loading, gpd/ft^2	9.1
Type RBC Drive	Air Drive
Media Submergence	40%
(Existing) Aeration Tank Volume, mg	2.00
(Existing) Aeration Tank Depth, ft	14
Hydraulic Retention Time (Design Flow) hrs.	2.40
Aerator Average MLSS	500-3000
Maximum Blower Capacity, cfm	8,500
Final Sedimentation Tank	
Number	1
Size - ft	238 Lx 70W
Area, ft.^{2}	16,600
Volume, MG	1,375
Weir Length, ft	146
Weir Overflow, gpd/ft	137,000
Surface Loading, gpd/ft^2	1,205
Settled Sewage Pumping Station	
Range of Capacity, mgd	10-30
Type Pump	Variable Flow
Return Sludge Pumping Station	
Range of Capacity, mgd	1-20
Type Pump	Variable Flow
Performance	
Expected Surfact Influent	
BOD_5, mg/l	160
SS, mg/l	130

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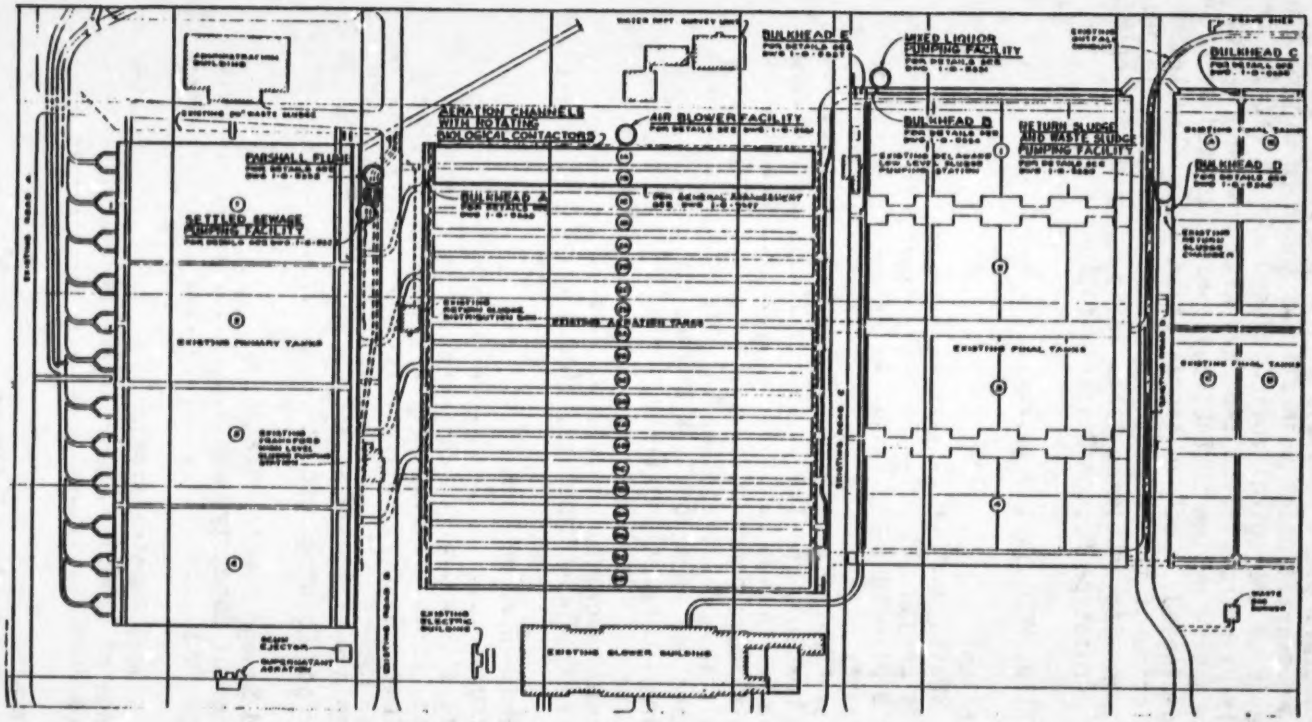


FIGURE 2

Final Effluent	
BOD_5, mg/l	19
SS, mg/l	30

A settled sewage pumping station provides a known controlled flow to the SURFACT process. In order to isolate SURFACT return sludge, a separate pumping station was provided.

Final design for this installation commenced late in 1976 and plans and specifications were advertised in mid-'77 and operation began mid-September '77. Figure 2 superimposes the prototype installation on existing Northeast plant. Because of simplicity of installation, construction time was 60 days. This confirmed our original thoughts regarding ease and speed of construction.

Construction Costs & Preliminary Treatment Economics

The applicable costs to the SURFACT process upgrading are:

Structures and Equipment	$ 422,000
Furnishing Contactors	908,000
Electrical	20,000
TOTAL	$1,350,000

Since 20 mgd will receive upgraded treatment, costs per mgd and additional BOD and SS may be computed. Effluent quality should increase from 56 to 19 mg/l for BOD and suspended solids from 60 to 30 mg/l. The additional pounds of pollutants removed resulting from this upgrade are 6200 lbs/day of BOD and 5000 lbs/day of suspended solids. Upgrade costs ratios are thus:

$67,500/MGD
$218/additional lb BOD removed
$270/additional lb SS removed

Prototype Operation

The system began operation on September 14, 1978. Debugging operations continued through late December. Most start-up problems related to return and waste sludge pumps. Virtually no structural or clogging problems with the rotating discs were observed.

Table 1 chronologizes prototype operation. The debugging period can be broken down into two phases - preliminary start-up and low SRT 12 mgd mode. Table 2 details prototype performance for the said periods. It must be remembered that during both phases substantial equipment malfunction occurred, thus preventing process stability. Throughout both phases disc rotation at 1½ rpm was no problem and no structural or clogging problems were evident. After start-up growth appeared on discs within 4 days and by 10 days average slime thickness of 40 thousandths of an inch was attained on the front discs.

Tables 3 - 10 outline system performance after debugging and stability

TABLE 1. CHRONOLOGY OF OPERATION

9/14 - 10/1/77	Preliminary Start Up
10/01 - 12/4/78	Low SRT Mode Flow 12 MGD Start-Up Problems
12/23 - 1/9/78	SRT 3.3 Flow 12 MGD
1/31 - 2/14/78	SRT 4.4 Flow 12 MGD
2/27 - 4/02/78	SRT 5.5 Flow 10 MGD
4/04 - 5/12/78	SRT 4.4 Flow 15 MGD
5/26 - 7/16/78	SRT 7 FLOW 10 MGD
8/4 - 8/13/78	SRT 1.7 Flow 10 MGD No Return Sludge

TABLE 2 SURFACT OPERATING PARAMETERS

Time Period: 9/14-10/1/77* Start Up & Debugging

Flow (mgd) 8.8

Plant Performance	SS	BOD	SBOD
Surfact Efficiency %	83	81	44
Surfact + Primary Efficiency %	92	82	
Effluent Quality mg/l	24	38	29

Time Period: 10/1-12/4/77* Start Up & Debugging

Flow (mgd) 12.0

Plant Performance	SS	BOD	SBOD
Surfact Efficiency *	59	69	74
Surfact + Primary Efficiency *	79	74	
Effluent Quality mg/l	60	59	23

* Many operational problems experienced throughout period

were achieved. Parameters used in these tables are defined in the appendix.

Mode I

The first mode, 12/23/77 through 1/9/78, was run at a moderate SRT (3.3) and a 12 mgd flow (see Table 3). Wastewater temperatures averaged 55°F and respectable secondary efficiencies were obtained. The low wastewater temperatures and the moderate SRT allowed dissolved oxygen levels to remain above 4.0.

Mode II

During the second mode wastewater flow, temperature and dissolved oxygen levels stayed approximately the same as the previous phase. The SRT was increased from 3.3 to 4.4 resulting in improved performance and lower sludge production. Refer to Table 4.

Mode III

In the third mode (2/27 - 4/2/78), the flow was dropped to 10 mgd and the SRT raised to 5.5 (see Table 5). Wastewater temperature and dissolved oxygen levels were similar to the first two phases. Secondary efficiencies mirrored the previous phase.

Mode IV

During the fourth mode (Table 6) flow was increased to 15 mgd. SRT dropped to 4.4 and wastewater temperature increased. The combination of all these factors contributed to lower dissolved oxygen levels than in previous modes. For the first time since system start-up difficulty was experienced in keeping D.O. levels continually above 2.0 at the aerator effluent. This problem was due to the inherent oxygen transfer capability of the existing coarse bubble diffuser system. During this mode chronic problems with return sludge pumps occurred. This resulted in undesirable fluctuations in mixed liquor solids concentrations and unreliable data for sludge production and total removals of BOD and suspended solids. Despite D.O. and return sludge problems, good soluble BOD removal (86%) was noted.

Mode V

Flow was returned to 10 mgd for mode five, SRT raised to 7, and wastewater temperature increased from 59 to 61°F (Table 7). Dissolved oxygen levels were more consistent than Mode IV. Secondary efficiencies returned to the high levels noted in Modes II and III. In fact, this mode maintained the highest BOD and SBOD efficiencies of any phase in the prototype work. BOD and SBOD removals were 91 and 95% respectively. Table 8 shows the daily performance of the system and emphasizes the consistent effluent quality obtained. At the later portion of the phase effluent quality approached tertiary. Since this phase carried the highest SRT of the study sludge production was at its low point (.69 lbs solids/lb BOD removed).

TABLE 3 SURFACT OPERATING PARAMETERS

Time Period: 12/23/77 - 1/9/78

MODE I

Operating Criteria

Flow (mgd)	12
Wastewater Temp.	55°F

	Primary	Aerator	Final
Detention Time (hrs.)	0.7	4.0	2.8
SOR gpd/ft^2			720
Solids Loading lbs/ft^2/d			12
MLSS mg/l		1496	
RBC Equivalent solids mg/l		1290	
SVI			61
Underflow Concentration mg/l			6583

Organic Loading Parameters

			With Disc	Without Disc
lbs BOD/1000 cf	68			
lbs BOD/ft^2 media	8.24	F/M	0.5	1.0
lbs SBOD/ft^2 media	3.75	SRT	3.3	1.8

Air

ft^3 /gal*	0.9	lb O_2/lb BOD Removed	0.5
ft^3 /lb BOD Applied	650	Average D.O.	4.5

Sludge Production

lbs/lb BOD Removed	.91

Power

KWH/lb BOD Removed	0.55

Plant Performance	SS	BOD	SBOD
Surfact Efficiency %	72	82	83
Surfact + Primary %	86	85	
Final Effluent mg/l	29	31	10

* Diffuser transfer efficiency 3%.

TABLE 4 SURFACT OPERATING PARAMETERS

Time Period: 1/31/78 - 2/14/78 MODE II

Operating Criteria

Flow (mgd)	12
Wastewater Temp.	52°F

	Primary	Aerator	Final
Detention Time (hrs.)	0.6	4.0	2.8
SOR gpd/ft^2			720
Solids Loading lbs/ft^2/d			22
MLSS mg/l		2948	
RBC Equivalent solids mg/l		1672	
SVI			65
Underflow Concentration mg/l			12539

Organic Loading Parameters

			With Disc	Without Disc
lbs BOD/1000 cf	74			
lbs BOD/ft^2 media	9.0	F/M	.34	.56
lbs SBOD/ft^2 media	4.4	SRT	4.4	2.8

Air

ft^3/gal*	1.2	lb O_2/lb BOD Removed	0.5
ft^3/lb BOD Applied	700	Average D.O.	4.4

Sludge Production

lbs/lb BOD Removed	.91

Power

KWH/lb BOD Removed	0.55

Plant Performance

	SS	BOD	SBOD
Surfact Efficiency %	84	87	93
Surfact + Primary %	87	88	
Final Effluent mg/l	26	25	7

* Diffuser transfer efficiency 3%.

TABLE 5 SURFACT OPERATING PARAMETERS

Time Period: 2/27 - 4/2/78 MODE III

Operating Criteria

Flow (mgd)	10
Wastewater Temp.	56°F

	Primary	Aerator	Final
Detention Time (hrs.)	0.7	4.8	3.3
SOR gpd/ft^2			600
Solids Loading lbs/ft^2/d			14
MLSS mg/l		2105	
RBC Equivalent solids mg/l		1821	
SVI			63
Underflow Concentration mg/l			10744

Organic Loading Parameters

			With Disc	Without Disc
lbs BOD/1000 cf	57			
lbs BOD/ft^2 media	6.94	F/M	0.30	0.75
lbs SBOD/ft^2 media	3.85	SRT	5.5	2.7

Air

ft^3/gal*	1.2	lb O_2/lb BOD Removed	0.6
ft^3/lb BOD Applied	870	Average D.O.	5.0

Sludge Production

lbs/lb BOD Removed	1.0

Power

KWH/lb BOD Removed	0.70

Plant Performance	SS	BOD	SBOD
Surfact Efficiency %	84	87	93
Surfact + Primary %	89	89	
Final Effluent mg/l	23	23	7

* Diffuser transfer efficiency 3%.

TABLE 6 SURFACT OPERATING PARAMETERS

Time Period: 4/4 - 5/12/78 MODE IV*

Operating Criteria

Flow (mgd)	15
Wastewater Temp.	59°F

	Primary	Aerator	Final
Detention Time (hrs.)	0.8	3.2	2.2
SOR gpd/ft^2			900
Solids Loading lbs/ft^2/d			13
MLSS mg/l		1535	
RBC Equivalent solids mg/l		2013	
SVI			57
Underflow Concentration mg/l			10240

Organic Loading Parameters

			With Disc	Without Disc
lbs BOD/1000 cf	89			
lbs BOD/ft^2 media	10.8	F/M	.55	1.74
lbs SBOD/ft^2 media	4.9	SRT	4.4	1.92

Air

ft^3/gal**	0.8	lb O_2/lb BOD Removed	.42
ft^3/lb BOD Applied	510	Average D.O.	3.4

Sludge Production

lbs/lb BOD Removed	.70

Power

KWH/lb BOD Removed	0.46

Plant Performance	SS	BOD	SBOD
Surfact Efficiency %	67	76	86
Surfact + Primary %	81	78	
Final Effluent mg/l	42	44	12

* During mode substantial return sludge problems present.
** Diffuser transfer efficiency 3%.

TABLE 7 SURFACT OPERATING PARAMETERS

Time Period: 5/26 - 7/16/78 MODE V

Operating Criteria

Flow (mgd) 10
Wastewater Temp. 61°F

	Primary	Aerator	Final
Detention Time (hrs.)	1.0	4.8	3.3
SOR gpd/ft^2			600
Solids Loading lbs/ft^2/d			16
MLSS mg/l	2460		
RBC Equivalent solids mg/l	1453		
SVI			59
Underflow Concentration mg/l			9993

Organic Loading Parameters

			With Disc	Without Disc
lbs BOD/1000 cf	49			
lbs BOD/ft^2 media	6.0	F/M	.27	.43
lbs SBOD/ft^2 media	3.5	SRT	7.0	4.4

Air

ft^3/gal*	1.1	lb O_2/lb BOD Removed	.61
ft^3/lb BOD Applied	900	Average D.O.	2.8

Sludge Production

lbs/lb BOD Removed .69

Power

KWH/lb BOD Removed .65

Plant Performance	SS	BOD	SBOD
Surfact Efficiency %	82	91	95
Surfact + Primary %	93	92	
Final Effluent mg/l	18	14	5

* Diffuser transfer efficiency 3%.

TABLE 8

SURFACT - Daily Performance

MODE V Flow 10 mgd SRT 7.0

Date	Effluent Quality mg/l			Percent Removal Surfact			Surfact + Primary	
	SS	BOD	SBOD	SS	BOD	SBOD	SS	BOD
6/24/78	18	14	3	78	91	97	93	96
6/25/78	13	17	5	86	89	94	95	91
6/26/78	16	11	3	86	92	96	95	96
6/27/78	17	15	5	84	91	95	99	93
6/28/78	19	13	4	81	93	96	93	94
6/29/78	16	12	4	89	94	96	95	96
6/30/78	15	15	4	88	90	95	96	94
7/ 1/78	16	12	2	85	90	97	91	93
7/ 2/78	14	8	2	88	94	97	95	97
7/ 3/78	12	7	2	88	94	97	96	96
7/ 4/78	11	6	2	86	93	96	91	96
7/ 5/78	11	7	3	87	93	95	95	95
7/ 6/78	14	12	6	86	90	92	95	96
7/ 7/78	20	8	5	85	94	94	94	94
7/ 8/78	13	7	5	87	95	93	96	-
7/ 9/78	14	8	3	86	94	96	95	-
7/10/78	13	6	2	88	95	97	95	-
7/11/78	10	8	3	89	95	97	96	-
7/12/78	14	6	2	80	97	98	95	-
7/13/78	15	7	4	85	95	97	94	-
7/14/78	5	6	2	93	97	98	98	-
7/15/78	4	6	3	91	96	96	98	-
7/16/78	6	6	3	92	96	97	99	-

Mode VI

In the last mode (Table 9) the system was run with the same wastewater flow but no return sludge. Basically, this meant that the growths on the rotating biological contactors provided the only source of biomass. Wastewater temperature climbed to 66°F and with no return sludge dissolved oxygen levels averaged over 4. Even though the SRT dropped to 1.7 good treatment was provided. Table 10 illustrates daily performance for this mode.

On August 14, 1978 SURFACT prototype was shut down to perform modifications to the air diffuser network and sewage feed system.

Prototype Maintenance

As mentioned earlier, during the eleven months of prototype operation very minimal maintenance or operator attention was required. Mechanical headaches were almost exclusively generated by return or waste sludge pumps. Even in the advanced stages of the study it was not uncommon for major pump failure to occur. This resulted at times in undesirable variations in the mixed liquor solids wasting rates, etc.

The rotation speed for all 22 discs was set at 1½ rpm. For the most part, once set disc speeds would not vary. Occasionally, on several of the contactors, an imbalance or lobing would be observed. By increasing the amount of supplemental air to that disc its speed would increase and within a matter of several days the imbalance would cease. The only additional maintenance or operator functions for SURFACT is caring for the discs. This activity includes monitoring disc speed, balance and weight, checking supplemental air flow and lubricating shaft bearings. The level of effort varies with the frequency of monitoring and the level of instrumentation and is felt to be minimal.

Throughout the eleven month operation, no structural problems surfaced. Upon draining the aerators for modifcation work it was noticed that roughly 50 air cups had sheared off the discs. This represents less than 1% loss and did not alter the ability of the air drive system.

Necessary Modifications to Prototype System

Midway through the study it was noted that the existing diffused air system with an oxygen transfer efficiency of 3 percent could not support the oxygen demand of the system for flows exceeding 14 to 15 mgd and BOD removals in the 140 mg/l range.

The present system can transfer 6200 lbs/day of oxygen using an air flow of 8500 cfm. The discs add another estimated 3 lbs O_2/1000 ft^2 or 6600 lbs, bringing the total capacity to 12,800 lbs/day. At 20 mgd and BOD removal of 141 mg/l, 23,500 lbs of oxygen are required daily. This assumes 1.0 lbs O_2 are required per pound of BOD removed. To correct this limitation fine bubble ceramic diffusers are being installed in both SURFACT tanks. Transfer efficiencies for these domes are 13%, yielding 19,000 lbs O_2/day for the system using an air flow of 6000 cfm. Adding the 6600 lbs O_2/day from the discs

TABLE 9 SURFACT OPERATING PARAMETERS

Time Period: 8/4 - 8/13/78

MODE VI
No Return Sludge

Operating Criteria

Flow (mgd)	10
Wastewater Temp.	66°F

	Primary	Aerator	Final
Detention Time (hrs.)	1.1	4.8	3.3
SOR gpd/ft^2			600
Solids Loading lbs/ft^2/d			.61
MLSS mg/l		123	
RBC Equivalent solids mg/l		1417	
SVI			36
Underflow Concentration mg/l			743

Organic Loading Parameters

			With Disc	Without Disc
lbs BOD/1000 cf	42			
lbs BOD/ft^2 media	5.1	F/M	.55	8.28
lbs SBOD/ft^2 media	2.4	SRT	1.72	.13

Air

ft^3/gal*	0.5	lb O_2/lb BOD Removed	.29
ft^3/lb BOD Applied	410	Average D.O.	4.4

Sludge Production

lbs/lb BOD Removed	1.52

Power

KWH/lb BOD Removed	46

Plant Performance	SS	BOD	SBOD
Surfact Efficiency %	83	86	88
Surfact + Primary %	93	91	
Final Effluent mg/l	18	18	7

* Diffuser transfer efficiency 3%.

TABLE 10 SURFACT - Daily Performance

MODE VI Flow 10 mgd No Return Sludge

	Effluent Quality mg/l			Percent Removal Surfact			Surfact & Primary	
Date	SS	BOD	SBOD	SS	BOD	SBOD	SS	BOD
8/4/78	22	19	4	84	88	94	94	91
8/5/78	22	15	8	78	90	86	92	90
8/6/78	21	18	5	76	87	93	93	92
8/7/78	16	19	7	84	80	86	94	90
8/8/78	16	19	8	84	88	92	95	88
8/9/78	19	15	8	87	91	91	93	95
8/10/78	15	18	8	86	88	89	94	93
8/11/78	16	19	8	84	84	88	94	92
8/12/78	16	15	5	84	86	89	95	93
8/13/78	19	20	6	80	81	76	91	83

the total becomes 25,600 lbs. The installation scheduled for completion in mid-October will not only provide the additional oxygen but at reduced air flow-6000 versus 8500 cfm. Air flow reduction means less shear and energy consumption. The system also allows positive control of air flow at aerator tank quarter points which permits better air distribution and utilization. Once the oxygenation limitation is corrected higher flows can be sent through the prototype. In order to assure proper solids flux loading to the clarifier (lbs/ft^2/day) and uniform loading to the discs a step feed system will be constructed. The feed system will allow a maximum of four point wastewater feeds equally spaced down the tank. Step feed should be available in early '79.

General Discussion

SURFACT performance relates well to the activated sludge process. Normal relationships for process variables were similar for SURFACT and activated sludge. Figure 3 shows the relationship between SRT and % removal of BOD, soluble BOD and suspended solids. Points for mode IV are shown but not used because of excessive return sludge problems and D.O. fluctuations. The graph shows the expected trend - removal increases with SRT. As the SRT goes beyond 5 leveling occurs. From the graph it is evident that soluble BOD removals were quite respectable (ranging from 83 to 95 percent) throughout the entire study. This consistent performance indicates a strong biological process with ability to produce a good effluent. As with the activated sludge process good solids separation in the final clarifier is the key to a quality effluent. Referring to Figure 3 as SRT approached and exceeded 5 suspended solids removals improved. Thus it may be possible to attain the soluble BOD removal at SRT's lower than 5 but to assure good solids and particulate BOD removals it will be necessary to average 5. Figure 4 illustrates the relationship between sludge production and SRT and F/M. Sludge production is calculated as follows:

$$\frac{\text{Waste sludge flow x waste sludge concentration}}{\text{lbs BOD removed}}$$

Although data is scattered the general trends agree with activated sludge theory. When F/M increases so does sludge production. The reverse is true for SRT. Some of the sludge production numbers require qualification. During modes I through IV contractual repairs in the primaries caused excessive downtime and poor effluent quality. Table 11 lists influent suspended solids for each mode.

Table 11

Mode	Surfact Influent* SS mg/l
I	115
II	163
III	153
IV	136
V	100
VI	107

* Note: Primary effluent is the influent to the SURFACT process

FIGURE 3

SURFACT % REMOVALS VS. SRT

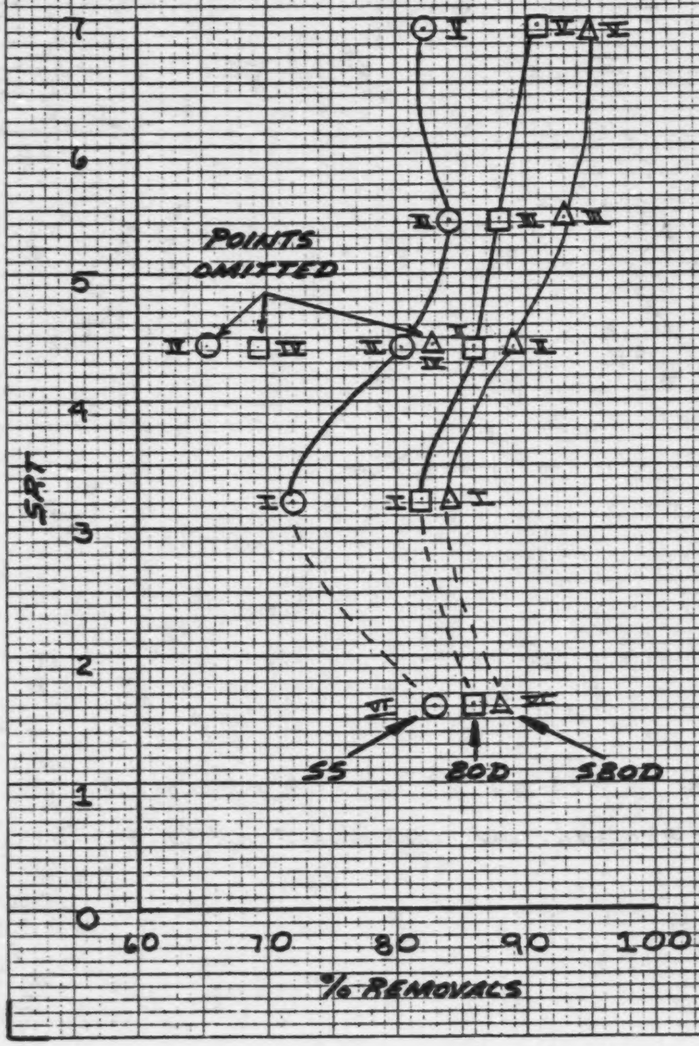
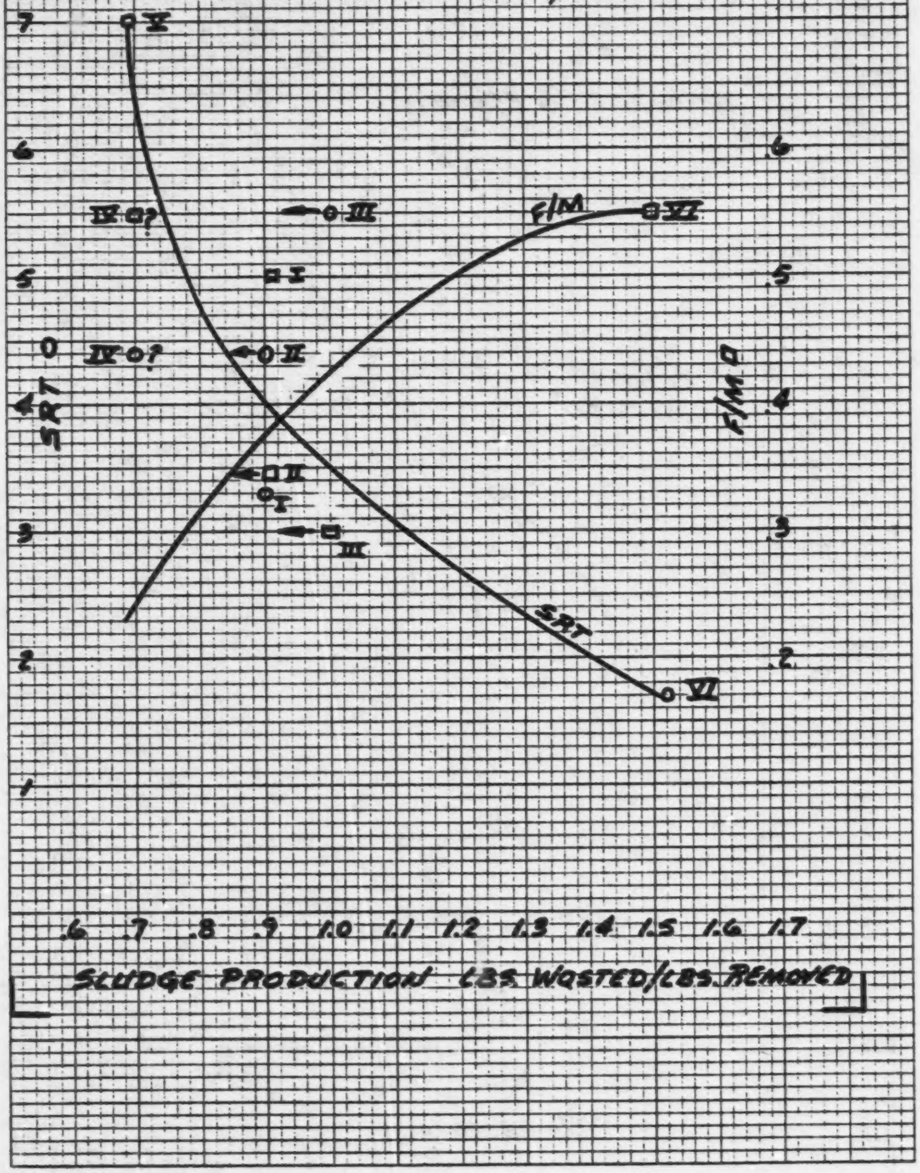


FIGURE 4

SURFACT

SLUDGE PRODUCTION VS. F/M & SRT



With good primary treatment a representative influent suspended solids level would be 110 to 120 mg/l. Therefore, modes II, III and IV are carrying and removing an abnormal level of primary solids which inflates sludge production. On Figure 4 arrows are placed adjacent to several of the data points showing the direction the point would shift if good primary treatment were available. It should be evident that the points in question, if shifted, would smooth out the curve.

Since SURFACT merges activated sludge with rotating biological contactors, the relationship between mixed liquor, disc solids and other process parameters requires exploration. Figure 5 illustrates the relationship between mixed liquor suspended solids, equivalent solids from the disc and soluble BOD loading on the disc for each study mode. Equivalent solids from the disc are calculated by taking the weight of solids on the discs and dividing same by aerator volume.

Equivalent disc solids =

$$\frac{\text{Dry weight of growth on 22 discs-lbs}}{\text{Aerator Volume (2 mg)} \times 8.34}$$

To assure "steady state" weight of the discs for each phase the last available weight for that phase was used rather than the average.

It was first thought that there would be a competitive relationship between the return sludge and the growth on the discs. As the mixed liquor increased, discs solids weight would decrease and vice versa. What is evident from Figure 5 is that a competitive relationship does not exist and that the disc solids weight varies only slightly from one mode to another while other parameters ranged appreciably. Stated another way the discs can be expected to perform a certain baseload of work regardless of the mixed liquor concentration. There does seem to be a relationship between the soluble BOD loading per square foot of disc versus disc solids weight. It appears that disc weight is relatively insensitive to mixed liquor solids variation and somewhat responsive to soluble BOD loading. Figure 6 confirms that a relationship for the said variables exists.

Although the weight of disc solids is fairly insensitive to many variables the distribution of weight from disc to disc along the tank length does change. Table 12 shows the thickness of the disc slime for all eleven discs in bay 1a for the different modes. Generally the thickness decreases the further downstream the disc. As influent loading changes and the mixed liquor concentration varies, disc thickness (which is comparable to weight) shifts. Figure 7 represents a "smoothed" profile of disc slime thickness for the entire tank length. Modes IV, V, and VI were selected for discussion purposes. If a classic profile exists it would be represented by mode V. Slime thickness sharply decreases between discs 2-4 and levels off after disc 5. For mode IV the higher soluble loading per square foot of media and the lower mixed liquor solids resulted in almost constant slime thickness for the length of the tank. During this mode it appears the full potential of the downstream discs were harnessed. More food was available to the later discs because of the higher soluble loading and the lower mixed liquor. In

FIGURE 5

SURFACT

EQUIVALENT DISC SOLIDS, MIXED LIQUOR SOLIDS, SOLUBLE LOAD VS. MODE

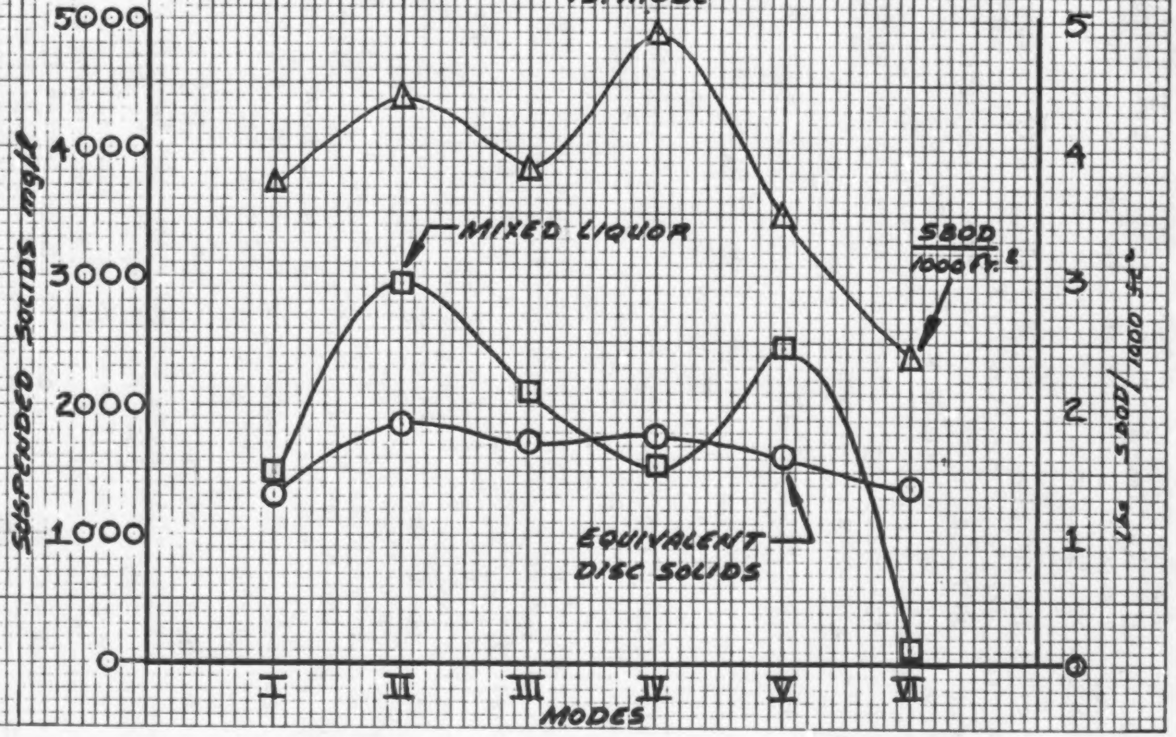


FIGURE 6 SURFACT

EQUIVALENT DISC SOLIDS VS. SOLUBLE LOADING

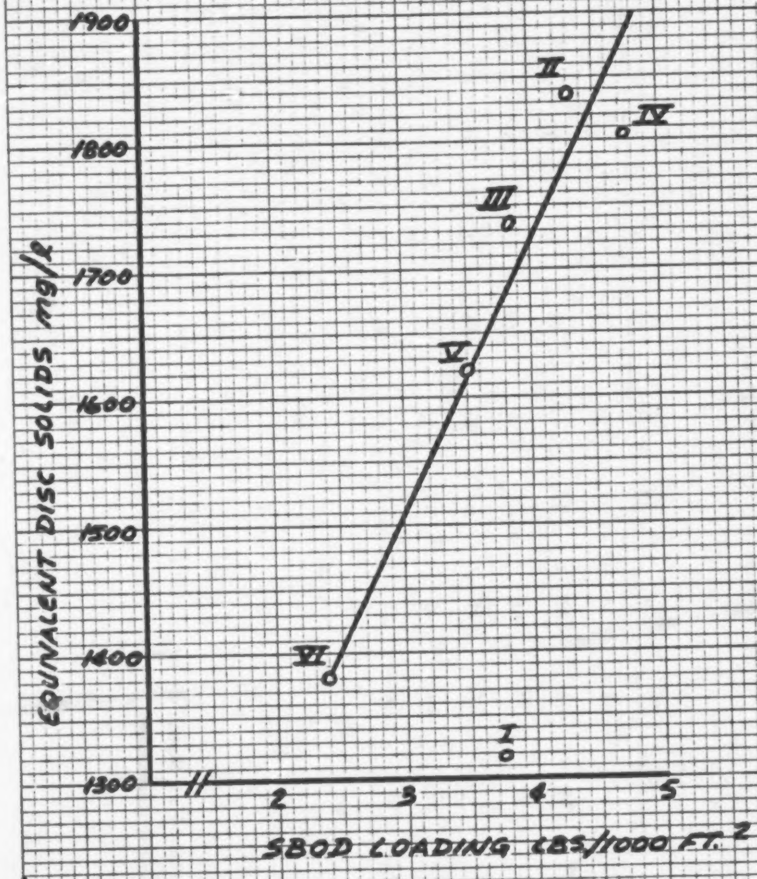


TABLE 12 Slime Thickness for Individual Discs

Disc	1a	2a	3a	4a	5a	6a	7a	8a	9a	10a	11a
MODE I	41*	41	37	31	27	27	21	21	23	21	22
MODE II	49	49	47	47	47	46	32	37	29	25	31
MODE III	55	70	66	62	62	70	64	55	23	43	39
MODE IV	53	57	55	53	57	55	55	58	57	58	47
MODE V	57	60	49	39	35	37	35	39	39	31	37
MODE VI	51	51	41	37	39	41	39	43	41	41	45

* Expressed as thousands of inch

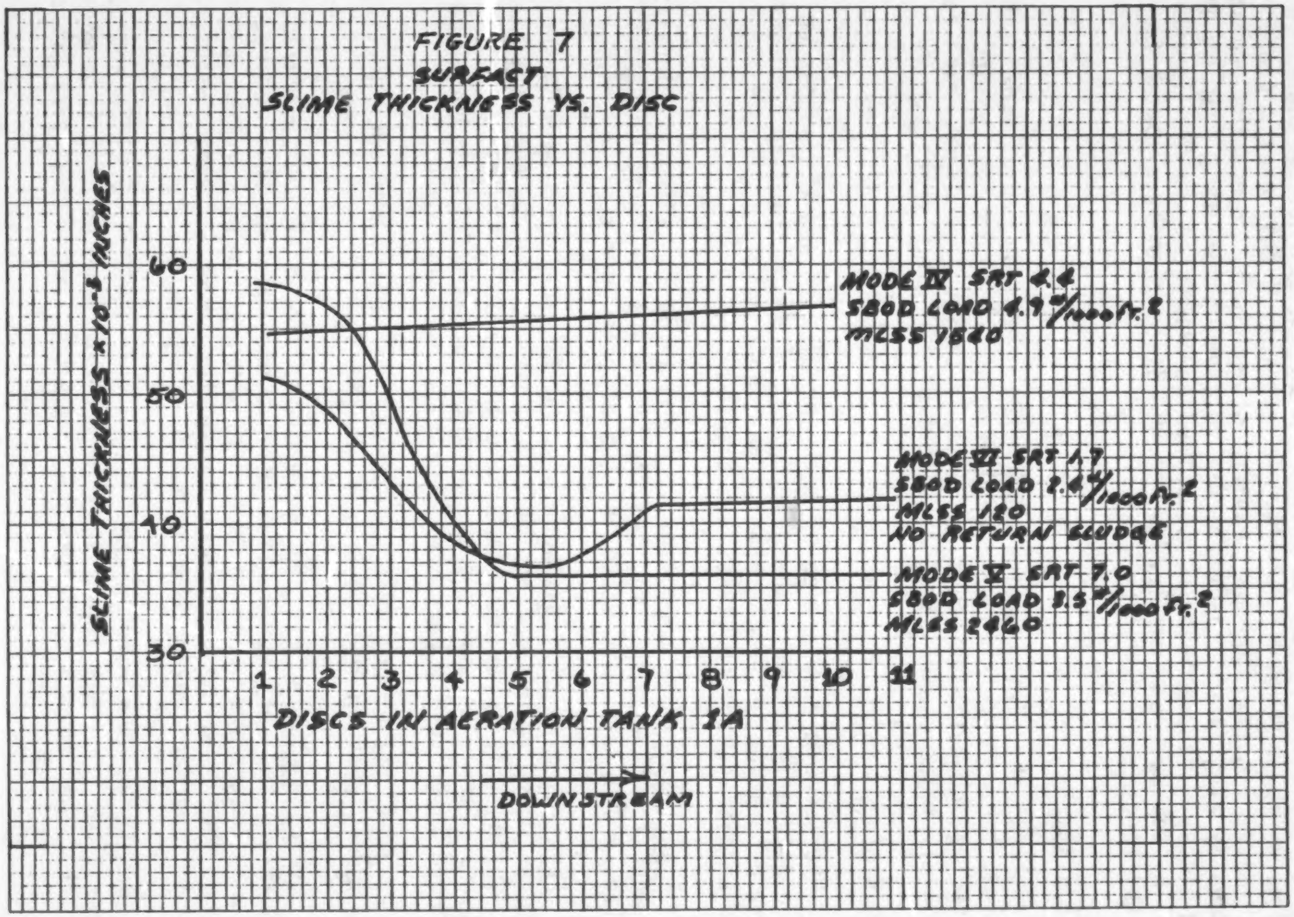


FIGURE 7
SURFACT
SLIME THICKNESS VS. DISC

mode VI no return sludge was employed. This resulted in little or no mixed liquor solids and a low SRT. Now since the discs are carrying the full load the downstream discs have thicker growth than mode V. Based on this discussion it is clear that to fully utilize the SURFACT system all discs should carry maximum growth levels. To accomplish this goal a proper match between mixed liquor, flow and loading is important. Since the prototype is presently plug flow it is difficult to develop maximum growth on downstream discs. The provision of step feed as mentioned earlier in the text will allow for more uniform loading to the discs thereby allowing for maximum slime thickness throughout the tank.

Special Studies

During the eleven months of SURFACT operation many special studies were conducted. These studies ranged from microbiology observations to establishing the relative reaction ratio of the suspended and fixed cultures. All special projects were aimed at understanding a new process.

Surfact Biology

Microscopic examination of both the biofilm and the suspended culture was made regularly to track the development of and changes in the microorganism population under different operating conditions. The following is a very brief discussion of SURFACT microbiology.

Flagellates were common or very common during all operational modes except when the flow was 15 mgd and the SRT was 4.4. There were more during this time, 4/4-5/12. This was so on the discs and in the liquors.

Free swimming ciliates were also common or very common during all operational modes except during the 15 mgd mode. They became rare in the liquors during this mode and became rare on the discs except for the last four discs. They also were reduced during the "no return sludge" mode but they seemed to be coming back when the system was shut down.

Single stalked ciliates were common to very common during all operational modes, both in the liquor and on the discs.

Branched stalked ciliates were common from start-up to 4/2/78, rare from 4/4-5/12, and very common from 5/26 to shut-down in the liquors. On the discs, they ranged from rare on disc 1 to very common on 11, except during the 15 mgd mode when they were only common from disc 9. They seemed to become more abundant during the no return sludge mode.

Nematodes were rare in the liquors from 12/23/77-5/26/78 and common at all other times. On the discs, they ranged from rare on disc 1 to very common on disc 11. They seemed to become less numerous during the no return sludge mode.

Rotifers were rare or absent in the liquors during all modes except V when they were common. They were common from midstream on the discs from start-up to 1/9. They were absent from 1/9-5/26. They were very common from

5/26-7/16. They were rare during the no return sludge mode but they seemed to be returning when the system was shut down.

Filaments, mainly Beggiatoa, were present but rare from start-up to May, became common during early June and continued to increase until shutdown. At no time was Beggiatoa in sufficient numbers to impact performance of the growth. It may be possible at wastewater temperatures greater than 66°F for their growth to interfere with performance.

Summary. Surfact is an aerobic system with flagellates, swimming ciliates and stalked ciliates common throughout. Nematodes and branched stalked ciliates were common from midstream. Rotifers were absent from January through May but were common on the later discs at all other times. Generally higher levels of treatment corresponded to higher life forms. Younger sludges had a predominance of flagellates free swimming and stalked ciliates while in the older sludges population shifts to branched ciliates, nematodes and rotifers were evident. All in all, SURFACT exhibited a fairly typical zoological make-up for an activated sludge system.

Rate Studies Biofilm versus Suspended Culture

For several of the modes oxygen uptake studies were performed. Warburg work for determination of k rates were run in mode V. The purpose of these studies was to determine the ratio of work done by the disc and the activated sludge. In table 13 oxygen uptake rates expressed as mg O_2/mg VSS and mg O_2/l plus k rates are tallied.

Columns 7 & 8 express gross oxygen uptake. Columns 5 & 6 prorate oxygen uptake relative to the weight of volatile solids present. The specific oxygen uptakes (cols. 5 & 6) are roughly equivalent for 2 of the three modes. In mode V the k rates developed using the Warburg apparatus were equivalent and confirm the specific oxygen uptake rates. Generally speaking the work done by the disc and suspended cultures is equivalent on a gram per gram basis. Therefore if the SURFACT system is run with equal mixed liquor and equivalent disc solids, work for both systems should be equivalent.

Soluble BOD Profiles

Soluble BOD was measured down the length of the reactor to determine the organic removal patterns affected by different operating modes. Data was collected during operating Modes V and VI. The data listed in Table 14 are mean values for the operating periods indicated.

Forty-four (44) percent of the profiles used to calculate the %SBOD removal for the SURFACT mode in Table 14 indicated that breakpoint occurred between the 1st and 6th disc; 56% of the profiles indicated breakpoint between the 6th and 11th disc. During Mode VI where no return was used all the profiles showed breakpoint occurring in the last half of the reactor. Figure 8 plots %SBOD removal vs Time and graphically depicts the difference in the removal patterns for the two operating modes. As expected without return sludge removals decreased.

TABLE 13

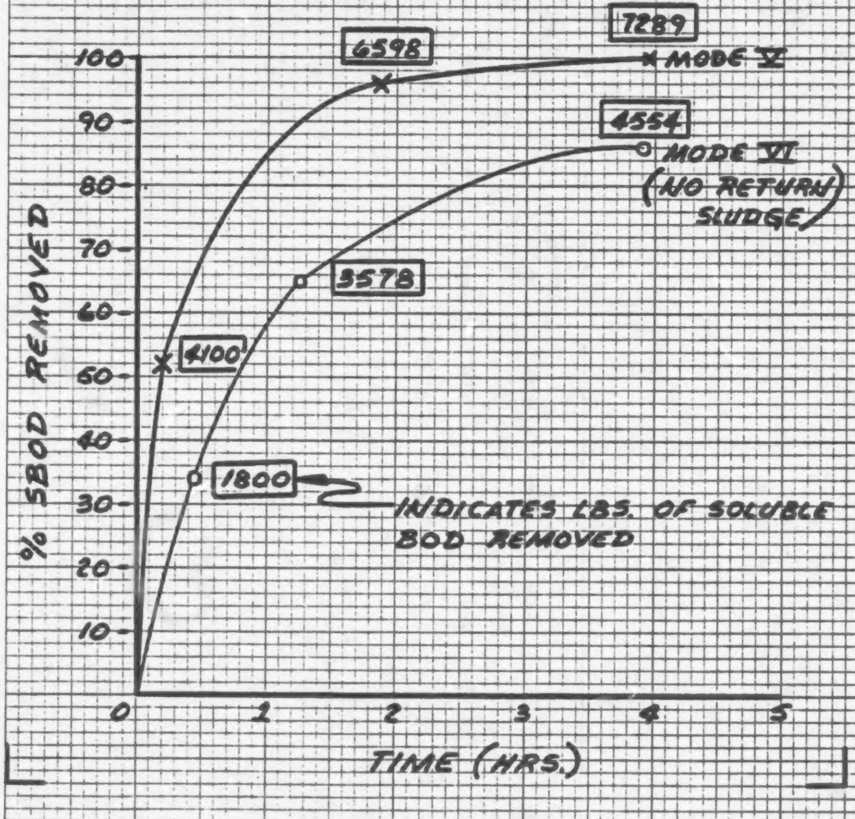
MODE	Flow	SRT	LBS BOD/1000 cf	Oxygen Uptake Ratio				K rate K_{10}	
				mg O_2/mg VSS/hr		mg O_2/l/hr			
				Suspended	Biofilm	Suspended	Biofilm	Suspended	Biofilm
III	10	5.5	57	.022	.022	35	33		
IV	15	4.4	89	.063	.048	76	78		
V	10	7.0	49	.039	.035	74	39	.160	.174

TABLE 14

Mode	Flow mgd	SRT Days	F/M	MLSS mg/l	RBC Equiv. MLSS mg/l	% SBOD Removal Beg.	Mid	End
V	10	7.0	0.27	2460	1450	54	86	95
VI	10	1.7	0.55	122*	1420	33	66	84

* No return sludge

FIGURE 8 SURFACT
% SBOD REMOVED VS. TIME



One salient characteristic of the removal patterns is the large initial decrease in SBOD between the influent and the first reactor sampling point. This decrease amounts to 54% SBOD removal and 33% SBOD removal for Modes V & VI respectively. For Mode V the retention time to the first sampling point is approximately 8 minutes; for the Mode VI where no return sludge was used the retention time is approximately 10 minutes. The pounds of soluble BOD removed in this short period averaged 4100 for Mode V and 1800 for Mode VI. This speaks to the system's ability to reduce soluble BOD in very short time intervals.

Settling Studies

Settling studies were run on SURFACT mixed liquor and return sludge to predict settling and overflow rates that would remove suspended solids adequately with a fixed underflow concentration. Tests were run for Modes III, IV and V. All data was treated by the Talmadge & Fitch method in which the area requirement for both clarification and thickening were determined and the larger area was used to calculate the allowable overflow and solids loading. Results of the settling tests indicate the SURFACT sludge settles well at high overflow rates and that the underflow concentration under such conditions will exceed one percent. Figure 9 illustrates the relation between mixed liquor solids concentration and initial settling velocity. The comparable overflow rate also appears in the graph. At a mixed liquor concentration of 4000 mg/l a settling velocity of 10 ft/hr would be expected. This corresponds to an overflow rate of 1800 gallons per square foot per day. Making these numbers more tangible since SURFACT at 20 mgd has a final tank overflow rate of 1200 gpd/ft^2 a mixed liquor solids level well in excess of 4000 mg/l could be accommodated. Since the discs can be credited with 1500 mg/l of equivalent solids it would be rare that the mixed liquor levels would approach 4000 mg/l.

Rotation of Discs

Throughout the prototype study the discs were maintained at 1½ rpm. The spiral roll existing in prototype tanks assists disc rotation. To ascertain the extent spiral roll assists in disc rotation air flow to the bottom diffusers was decreased and supplemental air increased to maintain 1½ rpm. Table 15 shows the relationship between spiral roll and supplemental air flow.

Table 15 Air Flow (cfm) To Maintain 1½ RPM

Bottom Diffusers	Supplemental Air	
	Per Disc	Total
8300	135	3000
3500	218	4800
0	297	6500

From the table it is obvious that if the bottom diffusers are shut off that a great deal more air flow (6500 vs. 3000 cfm) is required to maintain

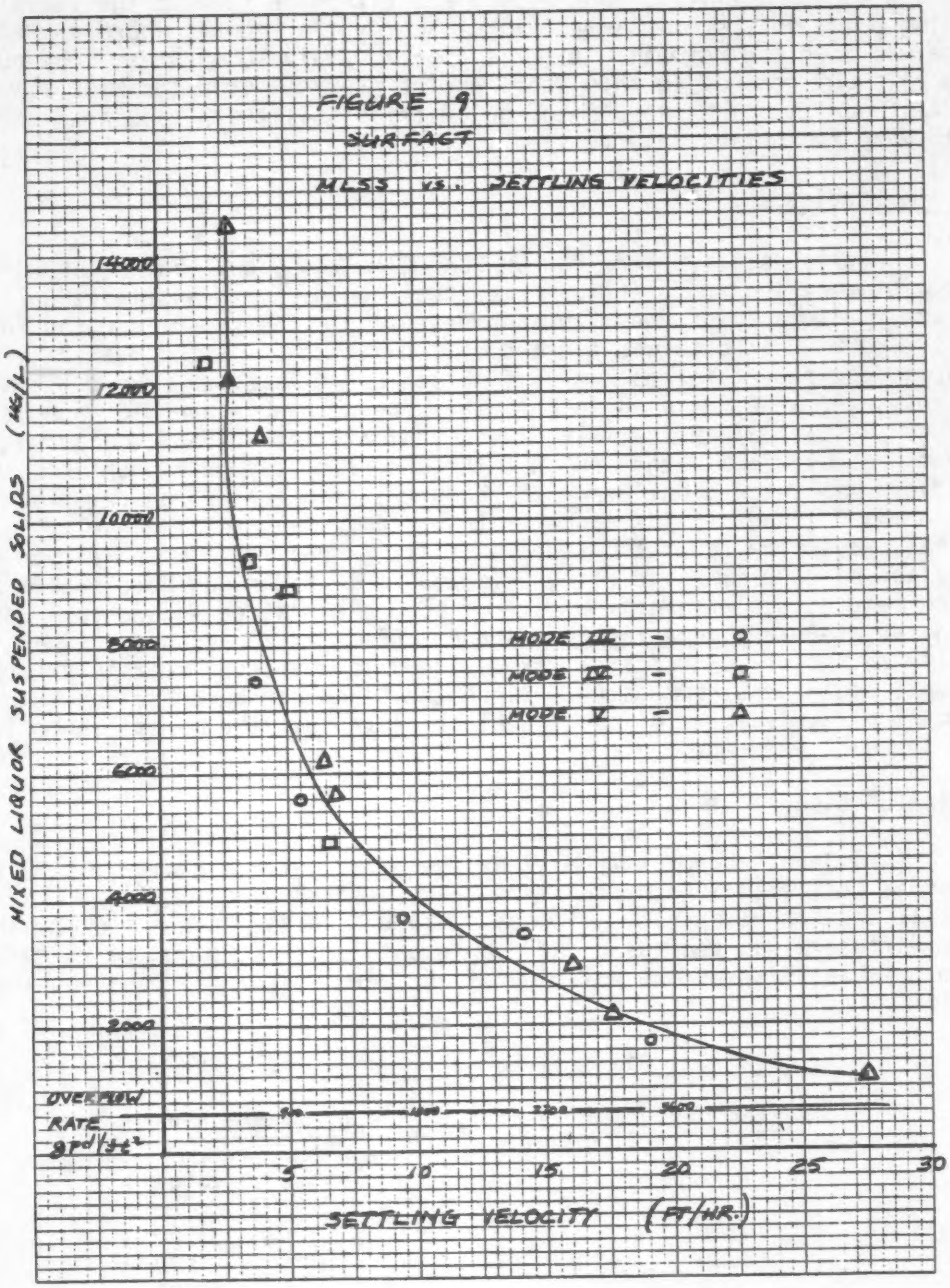


FIGURE 9

SURFACT

MLSS vs. SETTLING VELOCITIES

1½ disc rpm. This is an energy equivalent of 1100 kwh or 1.7 hp/mgd if a flow of 20 mgd is assumed.

One and one-half (1½) rpm may not be needed for all discs - for example higher speeds may be needed up front but slower speeds may be tolerable at later treatment stages. By dropping rpm from 1½ to 1 the air flow savings per disc is 100 cfm. Thus if rotation speed can be cut to 1 rpm for one-half the discs an air flow savings of 1100 cfm (22x½x100) would materialize. This calculates out to 400 kwh or .6 hp/mgd at a flow of 20 mgd.

Power Consumption Present-Future

Table 16 relates the power consumption expressed as kwh/lb BOD removed to the various modes and their respective air flows. The kilowatt hours used in this calculation are only for blower power both auxiliary and main and do not include return sludge pumps or miscellaneous motors or lights. Auxiliary air is discharged 4 ft. below water while main diffuser air is released at 12 ft. Therefore, the power consumption/cfm varies as follows:

Main Diffusers	.77 kwh/cfm
Auxiliary Air	.37 kwh/cfm

In Table 16 power consumed per pound of BOD removed generally relates inversely to flow with the exception of mode VI. During this mode lower air flows for the bottom diffusers were possible since no mixed liquor solids were present. By reducing air flow to the bottom diffusers a lower power contribution resulted. Noteworthy is the increase in auxiliary air flow for mode VI. By reducing bottom diffuser air flow the rotational force of spiral roll imparted on the disc was similarly reduced. In order to maintain 1½ rpm the auxiliary air was increased. This situation further confirms the contribution of spiral roll to disc rotation.

Power consumed/lb BOD removed in Table 16 probably could have been reduced if dissolved oxygen levels were not permitted to average over 2.0 mg/l. Four of the six modes had average D.O. concentrations in the 4 to 5 mg/l range and no mode averaged less than 3.8 mg/l. In future modes greater emphasis will be placed on trimming air flow to maintain a pre-selected D.O. level and power consumption should decrease.

As discussed earlier the installation of fine bubble diffusers will also reduce process power consumption. Assume for good treatment 6000 cfm of blower air and 4000 cfm of auxiliary air is required. Power consumption for the two air sources is:

$$6000 \text{ cfm} \times .77 + 4000 \text{ cfm} \times .37 \quad = \quad 6100 \text{ kwh}$$

while the lbs of BOD removed is:

$$20 \times (160-19) \times 8.34 \quad = 23{,}500$$

making the ratio kwh/lb BOD removed 0.26.

TABLE 16 Surfact Power Consumption

Mode	Diffused Air cfm	Auxiliary Air cfm	Flow mgd	D.O. mg/l	KWH/lb BOD removed
I	7569	2916	12	4.5	.55
II	9583	3055	12	4.4	.55
III	8680	2708	10	5.0	.70
IV	8125	3055	15	3.4	.46
V	7916	2916	10	2.8	.65
VI	2986	4930	10	4.4	.53
Future Mode with Dome Diffusers	6000	4000	20	2.0	.26

For mode V excellent treatment was provided for 22 hp/mgd. When fine bubble diffusers are installed the same level of treatment can be provided for 9.5 hp/mgd which is significantly lower than comparable secondary processes.

In a previous section entitled "Rotation of Discs" it was suggested that if ½ of the systems discs could be rotated at 1 versus 1½ rpm a savings of 1100 cfm of auxiliary air could be realized. This would further reduce power consumption below the 9.5 hp/mgd level.

This number should be approached as a goal. There may be other techniques to reduce power consumption such as configuring the bottom air diffusers in a way to eliminate the need for supplemental air entirely. Future work will address various fine tuning techniques for power reduction.

Costs

Upgrade

As reported earlier original costs for the SURFACT installation were $1,350,000. The installation of dome diffusers ($410,000) and step feed ($180,000) raises the construction total to $1,940,000. Since 20 mgd will be treated, and as noted previously, additional daily pollutant removal will be 6200 lbs. of BOD and 5000 lbs. of suspended solids, the following upgrade costs are calculated:

$97,000 mgd

$ 313/additional lb BOD removed

$ 388/additional lb SS removed

Although comparisons are difficult when relating upgrade costs the above costs are felt to be low since no tank, blower or other construction was necessary to provide better treatment.

New Construction

Recently a cost study was prepared which addressed construction of a SURFACT installation at the Northeast plant which would accommodate the 1990 flow of 250 mgd. Table 17 outlines the study results. The table lists construction costs as well as annual costs for debt service, maintenance and power. Element breakdowns for general and sludge are included to show the relationship of secondary versus total plant costs. For the secondary portion of the plant all items listed (construction cost, debt service, maintenance and power) were lower than comparable treatment systems. The first and second cost for SURFACT, whether as an upgrade or new construction, appear attractive.

Results at Full Load

The majority of study work for the past eleven months has not been at full load conditions (20 mgd). As mentioned in the text, oxygen limitation

Table 17 Construction & Annual Costs for 250 mgd Facility
Northeast Water Pollution Control Plant

Element	Capital	Debt Service*	Maintenance*	Power*
General	98.911**	8.742	1.436	.871
Secondary Treatment Using Surfact	59.250	5.664	.890	1.300
Solids & Gas Handling	96.190	7.338	1.219	.667
TOTAL	254.351	21.744	3.545	2.838

* Costs are annual

** All numbers are in millions of dollars problems prevented operation under design flow conditions. After the new diffuser system is installed along with the step feed piping the system will undergo full load testing. Based on the knowledge gained about the process in the past eleven months anticipated performance at 20 mgd will conform to design criteria. More specifically effluent quality for BOD and suspended solids will be 19 and 30 mg/l respectively.

Extrapolation from 10 to 20 mgd is possible because SURFACT behaves as an activated sludge process. At an SRT greater than 5 and an F/M of in the 0.3 range excellent treatment will be provided by activated sludge or SURFACT. The existing prototype system replete with its modifications can support mixed liquor solids in excess of 4000 mg/l and provide good solids separation in the final clarifier. Because SURFACT sludge settles well overflow rates well in excess of 1200 gpd/ft^2 can be tolerated. In addition the equivalent disc solids of 1500 mg/l will allow lower mixed liquors to be carried thus reducing the solids flux load to the final settlers.

Conclusions

Most importantly the study work in Modes I through VI indicates that SURFACT and activated sludge behave similarly. Advantages attributable to SURFACT mentioned earlier in the text have been confirmed through the study work.

The discs support a growth which is equivalent to 1500 mg/l of mixed liquor solids. These solids perform the same work as those in suspension on a gram per gram basis, obtain their oxygen in the same manner as conventional rotating contactors thus not depleting the oxygenation capacity of the aerator and do not present a solids load to the final settler. Essentially 1500 mg/l of mixed liquor solids are provided by the process without the associated oxygenation or settling liabilities.

The microbiology make-up of the disc solids and the suspended culture are for the most part identical. Microbiological population distribution is the same as activated sludge for comparable process conditions.

Energy costs are low because the disc solids which may represent 1/3 of the total system solids are self sustaining from an oxygen supply standpoint. Secondly, the discs can harness some of the hydraulic motion from the spiral roll thus saving rotation energy. From a maintenance and operational standpoint the system is no more complex than air activated sludge and operator attention is minimal.

Sludge settling characteristics for SURFACT are excellent. Initial settling velocities of 19 ft/hr were not uncommon. Throughout all study modes SVI's remained in the 60 range with little variation.

Oxygen limitations in the present prototype were discovered and are presently being corrected. Step feed will also be added to minimize solids overloading to the final clarifier and spread the organic load to all discs.

Construction time for upgrading situations is minimal and costs are low because no tank or equipment other than the discs is involved. Construction of new secondary facilities utilizing SURFACT also appears to be an attractive alternative regarding first and second costs.

Future Work

Philadelphia will subject SURFACT to full load conditions after all modifications are complete. In addition the impact of diurnal flows will be documented. Heretofore all work has been accomplished at steady flow conditions. Further documentation will be obtained in regards to sludge production. Values noted during past modes are higher than literature values for comparable systems.

One tank nitrification did not occur during the study period. This was primarily due to insufficient time during modes with the lowest F/M to develop a nitrifying culture. Although none of Philadelphia's expanded plants call for nitrification at present, future standards may dictate nutrient removal. As such future modes will pursue the ability of SURFACT to nitrify. Since the new diffuser system will be installed as spiral roll in one bay and full floor no spiral roll in the other, more information can be obtained regarding the energy credits or debits associated with spiral roll and disc rotation. Lastly more efficient techniques to utilize the diffused air to turn the discs will be researched. Most of the above work is in the category of "fine tuning." The process works well as is but more can be learned.

APPENDIX

PARAMETER DEFINITION

Detention Time hrs.	based on wastewater flow
SOR gpd/ft^2	based on wastewater flow
Solids Loading lbs/ft^2/day	based on wastewater and return sludge flow
RBC Equivalent solids mg/l	based on dry weight of disc solids divided by aerator volume
Ft^3 air/gallon	based on wastewater flow. Air flow does not include auxiliary blower used to drive discs.
Ft^3 air/lb BOD applied	does not include auxiliary blower used to drive discs.
Lb O_2/lb BOD removed	Lb O_2 calculated by assuming 3% transfer efficiency for diffused air. Auxiliary air not used in calculation Lb O_2 transferred = .641 x million ft^3 air applied
Lbs sludge/lb BOD removed	based on total suspended solids not volatiles Lbs sludge generated equals material wasted plus
Kwh/lb BOD removed	power includes diffused air and auxiliary blower but does not include return sludge
D O.	dissolved oxygen levels in mg/l are the average of grab aerator effluent samples for the day

Pretreatment Standards:
EPA Strategy to Curb Hazardous Industrial Discharges

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ABSTRACT

Toxic industrial wastes will have to be removed from wastewaters going into municipal sewer systems under a new EPA regulation (June 26, 1978) which has a major impact on many industries.

Affected are some 40,000 industrial plants in 21 categories which will be required to pretreat their discharged wastewaters using the best available technology economically achievable (BATEA) to remove 65 toxic chemicals and compounds.

These 65 chemicals interfere with or pass through normal sewage treatment operations or contaminate the sludge and effluents so that they cannot be recycled for agricultural purposes.

The new regulation does not specify numerical standards limiting concentrations. These standards will be set for each of the 21 industrial groups between now and December 1979. It is intended to aid local sewage treatment facilities in handling toxic substances, some of which are known to cause cancer. The pressure for safe waste management will push the process toward resource recovery and conservation while protecting the public health. Industries will have three years from the date the standards are set to comply.

Among the toxic chemicals covered are carbon tetrachloride, chloroform, cyanide, lead, PCB's, vinyl chloride and chlorinated organics. Eight industries are already covered by interim regulations, the other thirteen will be.

The industrial pretreatment standards under the EPA rules will be enforced primarily by local water treatment agencies. Localities must have enforcement programs in place by 1983 for industries served by their treatment plants which can handle one million or more gallons per day of wastewater.

The EPA estimates that it will cost the 568 cities and towns likely to be affected about $17 million over the next 5 years to set up pretreatment programs, while states will have to spend about $14 million.

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BACKGROUND

At the 5th Joint U.S.-Japan Conference on Sewage Treatment I discussed industrial wastewater pretreatment, the advantages of joint treatment, institutional relationship, synergistic, antagonist and acclimation factors.

Today I will describe the general pretreatment regulation that the Environmental Protection Agency (EPA) promulgated and published in the Federal Register on June 26, 1978. Also to be covered will be the effects on industrial discharges to Publicly-Owned Treatment Works (POTW), the options, basis for the standards, economic impacts and costs to industry.

EPA Deputy Administrator, Barbara Blum, said in announcing the regulations, "This reliance on local authorities helped by Federal financial assistance utilizes knowledge of the local situation to allow for flexible enforcement. Until a city can do its own enforcing, however, EPA and the states will enforce the new standards."

HIGHLIGHTS

By the end of 1982, about 40,000 U.S. industrial plants will have to install billions of dollars worth of wastewater treatment equipment to remove toxic pollutants before sending their wastewater into municipal sewage treatment plants. The EPA regulation is aimed at eliminating toxic chemicals that usually pass through municipal plants and on into the waterways. Pretreatment will also alleviate some of the process control problems that some cities are having with their treatment plants.

Toxic industrial chemicals in sewer systems can cause a number of potentially serious health and environmental problems. Once in the waterways, many of these pollutants are toxic to aquatic life, are long-lasting, can concentrate in the food chain, and are known or suspected cancer-causing agents. Among the high-volume chemicals industry must now remove are benzene, carbon tetrachloride, acrylonitrile, cyanides, and most heavy metals.

Industrial plants that discharge wastewaters directly into streams, lakes, and rivers already are being required to install treatment equipment. The new regulation tells industries in the United States that now discharge wastes (about 600 municipal plants) to do the same.

The regulation requires 21 industries, including electric utilities, iron and steel manufacturers, refineries, chemical process plants, pulp, paper and timber products plants, and machinery manufacturers, to install the best available technology economically achievable (BATEA). See Table I. The industries and pollutants were decided upon in a 1976 legal consent decree settling a lawsuit brought against EPA by the Natural Resources Defense Council.

Eight of the 21 industries have already been following EPA interim pretreatment guidelines, but 13 categories will have standards to meet for the first time. Additional industries and pollutants may be added later.

Industries will have three years from the publication date of the detailed regulations to comply. Local authorities will be responsible for enforcing the pretreatment standards, backed up by the Federal government and the states. Cities, which must develop enforcement programs for municipal plants handling more than 5 mgd (20,000 cubic meters) no later than 1983, will have ample incentive to monitor compliance. EPA has made Federal financing of new municipal plants--through its authorized construction grant program, which is now a total of $43.5 billion--contingent upon local acceptance of enforcement.

This reliance on local authorities, helped by Federal financial assistance, utilizes knowledge of the local situation. Until a city can do its own enforcing, EPA and the states will enforce the new standards.

EFFECTS OF INDUSTRIAL DISCHARGES TO POTW's

Industrial discharges to POTW's are known to be the source of significant problems. A number of the pollutants discharged by industrial users of POTW's are substances for which there is evidence of carcinogenicity, mutagenicity, and/or teratogenicity. Others are known to have acute toxic effects on human or aquatic organisms at sufficiently high concentrations. Many of the toxic pollutants are persistent in the environment and some bioaccumulate and enter food chains. When industrial pollutants enter POTW's they can create three types of problems:

(i) Interference. The most immediate impact of these pollutants can be on the operation of the POTW. Discharges of high volumes or concentrations of certain pollutants can inhibit or interfere with the proper operation of a POTW.

(ii) Sludge Management. Toxic pollutants, partially treated by a POTW, enter the POTW's sludge and can contribute significantly to sludge management problems. Industrial pollutants, particularly metals and other toxic pollutants, can limit the sludge management alternatives available to the POTW and increase the cost to the public of providing adequate sludge management. In some cases, improper handling of sludges contaminated with metals and other toxic pollutants can result in uptake of these pollutants by crops in the human food chain or leaching of these pollutants into ground water.

(iii) Pass-through. Even when the inhibition/interference and sludge management problems mentioned above have been dealt with, there still are many toxic industrial pollutants that do not receive adequate treatment in most POTW's. These toxic pollutants pass through POTW's in quantities and concentrations that can be harmful to the environment and that would be unacceptable under Federal, State, and local regulations dealing with industrial discharges directly to receiving waters. The pass-through of toxic industrial pollutants can prevent the attainment of water quality standards and increase the cost to consumers of treating drinking water.

THE OPTIONS

There are four options proposed which incorporate different concepts. This discussion will focus primarily on the major points of differences:

1. Option I--Local Enforcement of Technology Standards. Establish technology-based pretreatment standards for 21 industrial categories covering most incompatible pollutants, including 65 toxic pollutants. See Table II. Allow modification of categorical pretreatment standards for removal which the POTW is designed to achieve as well as for those removals which occur incidental to the treatment process, but only where the POTW implements a pretreatment program.

2. Option II--Local Enforcement of Technology Standards or Water Quality Variances. Establish pretreatment standards and provide for modifying standards as in Option I, but where approved, allow POTW's to establish and enforce substitute pretreatment requirements based on local water quality instead of the national pretreatment standards. Local enforcement of pretreatment standards wherever a pretreatment program is approved; Federal or NPDES State enforcement everywhere else.

3. Option III--Local Enforcement of Toxic Technology Standards. Establish technology-based pretreatment standards (and provide for modifying standards as in Option I) only for the most toxic pollutants (i.e., less than 21 industrial categories and 65 toxic pollutants). Allow POTW's to regulate all other pollutants based on guidance issued by the EPA and incorporate those limits in POTW's permit. Local enforcement of pretreatment standards wherever a pretreatment program is approved; EPA or NPDES State enforcement everywhere else.

4. Option IV--Federal/State Enforcement of Technology Standards. Establish technology-based standards for most if not all dischargers of incompatible pollutants (more than 21 industrial categories). Standards

to cover most, if not all, incompatible pollutants and modifications of standards allowed only for removal the POTW is designed to achieve. All enforcement of national pretreatment standards by EPA or NPDES States.

BASIS FOR PRETREATMENT STANDARDS

All four policy options presented for public comment provided for technology-based national pretreatment standards. However, one option enabled local authorities to develop and enforce locally-derived pretreatment limits in lieu of the national standards, provided that State Water Quality Standards or section 304(a) water quality criteria were not violated.

Proponents of pretreatment standards based on water quality in the receiving waters argued that such standards ensure that the dollar and energy costs of treatment do not exceed what local water quality conditions may require. Proponents assert that such standards would focus Federal and State resources on municipal discharges which directly affect the environment rather than on the many industrial sources whose environmental impact may be modified by treatment at the POTW. It was suggested that as technology improved technology-based controls provided an uncertain moving target for pretreaters.

Proponents of pretreatment standards based on the technological capabilities of industries emphasized that such standards provide for maximum progress towards the Act's basic goal of eliminating the discharge of pollutants into waters of the U.S., insure greater equity between industrial users of POTW's and direct dischargers who are already required to comply with technology-based standards, and will result in greater environmental protection since compliance with technology-based standards is far easier to determine.

In establishing its pretreatment policy the EPA has rejected the concept of national pretreatment limits based upon water quality considerations and will instead establish technology-based standards. Congress intended the pretreatment standards to be technology-based since "The Administrator shall, from time to time, as control technologies, processes, operating methods, or other alternatives change, revise such standards following the procedure established by this subsection for promulgation of such standards." And section 307(bx3) requires the pretreatment standards to be applied to categories of point sources. The EPA agrees with the need for technology-based standards, most significantly because many toxic industrial pollutants are conservative in nature, do not degrade in receiving waters, concentrate in bottom sediments and are thus not acknowledged in water quality measurement, and by virtue of their persistence and bioaccumulation can concentrate downstream in food chains and water supplies.

SUMMARY OF ECONOMIC IMPACT

The cost of implementing will be distributed among POTWs, NPDES States, and industrial users of POTWs. Only POTWs with a design flow of greater than 20,000 cubic meters will be required to develop pretreatment programs. Approximately 568 POTWs will be required to develop pretreatment programs. The regulation will affect approximately 40,000 industrial users of POTWs. Thirty NPDES

States and Territories will be required to administer State pretreatment programs. The total costs borne by each of these three sectors over the next 5 years is estimated on Table III.

A. Municipal Costs. Only POTWs with design flows of more than 20,000 cubic meters of water per day will be required to develop pretreatment programs. These POTWs will incur costs to develop the pretreatment programs, to operate the programs, and, in some cases, to apply for and maintain authorization to modify categorical pretreatment standards for pollutants removed by the POTW.

Federal funds are available to cover 75 percent of the costs of developing POTW pretreatment programs, and the costs here are net of this funding.

The regulation requires municipalities to develop adequate sources of revenue for the implementation of their pretreatment programs. In practice, this means that local costs will be distributed among users of the systems. If user charges are employed, both domestic and non-domestic users will support POTW costs. If industrial cost recovery or metered surcharges are employed, all municipal costs of pretreatment programs will be distributed among industrial users at an average of about $401 per discharger by 1983.

In 1983 municipal costs are highest. Average costs for POTW's sized 60 to 130 thousand/cwm/day in 1983 is $42,475 (average size of 568 POTW's is 110,000 CuM/day). For activated sludge treatment systems, the average total annual cost for a system with a flow of 80 thousand/CuM/day is $2,172,000.

B. NPDES State Costs. This cost declines as the number of local programs increases, but EPA estimates that about 1,900 POTW's sized 20 thousand/CuM/d or less receive industrial effluent. If the smaller POTW's do not develop their own programs, then pretreatment program responsibilities in these municipalities will continue to be performed by the NPDES States (and EPA).

The only costs shown by the analysis for 1978 is the cost of State enabling legislation and program development. This comes to about $54,000, or about $1,800 per State.

C. Industry Costs. Costs to industry as a direct result of this regulation are expected to be small. The regulation does require that dischargers monitor their own effluent at the time of promulgation of pretreatment standards applicable to them. Dischargers must also sample and analyze their effluent twice a year after the deadlines for compliance have been passed. The average annual costs per industrial user in 1983 may be approximately $2,900 and the total industry costs in 1983 may be $116,800,000; these costs include sampling, monitoring and reporting.

CONCLUSIONS

(1) Toxic industrial wastes will have to be removed from wastewaters going into POTWs.

(2) About 400,000 industrial plants in 21 major industrial categories will be required to use BATEA to remove 65 toxic chemicals.

(3) The new regulations do not specify numerical standards limiting concentrations of effluents at this time.

(4) Numerical limits or standards will be set by the EPA for each of the 21 industries by December 1979.

(5) Industries will have three years from the date the standard is set to comply with the regulation.

(6) Eight industries (electroplating, inorganic chemicals, leather tanning, nonferrous metals, petroleum refining, steam electric power, textile mills and timber products are already covered by interim pretreatment regulations. Thirteen others will be.

(7) It is estimated by EPA that it will cost the 568 cities and towns likely to be affected about $17 million over the next 5 years, while states will have to spend about $14 million. Industries will have to sample and test their effluents at an estimated cost of $117 million ($2,900 per plant).

The pretreatment regulation did not surprise industry because it was very close to the proposed strategy EPA published last year.

The EPA is protecting not only the environment but also people as well. The word "environment" is frequently too narrowly defined and is taken to mean only natural, inanimate objects or animal life, when really all that EPA is doing is the protection of human life.